CONSOLIDATION SETTLEMENT RESPONSE OF REINFORCED GRANULAR FILL-SOFT SOIL SYSTEM

By

ANIRBAN MANDAL

TH
CE/1999/M
M312C

DEPARTMENT OF CIVIL ENGINEERING
INDIAN INSTITUTE OF TECHNOLOGY KANPUR

August, 1999

CONSOLIDATION SETTLEMENT RESPONSE OF REINFORCED GRANULAR FILL-SOFT SOIL SYSTEM

A Thesis Submitted
in Partial Fulfillment of the Requirement
for the Degree of
MASTER OF TECHNOLOGY

By
ANIRBAN MANDAL

to the
DEPARTMENT OF CIVIL ENGINEERING
INDIAN INSTITUTE OF TECHNOLOGY, KANPUR
August, 1999

2 MAR 2000 CE
CENTRAL LIBRARY
I. I. T., KANPUR
A 130459
A130459

POST GRADUATE OFFICE
I.I.T. KANPUR
Submitted on 4.8.99

CERTIFICATE

This is to certify that the work contained in the thesis entitled " **CONSOLIDATION SETTLEMENT RESPONSE OF REINFORCED GRANULAR FILL-SOFT SOIL SYSTEM**" by **ANIRBAN MANDAL,** has been carried out under my supervision and this work has not been submitted elsewhere for a degree.

SARVESH CHANDRA

Professor

Department of Civil Engineering

Indian Institute of Technology

Kanpur-208016, INDIA

August, 1999

ABSTRACT

ANIRBAN MANDAL

Roll No. 9710304 (M.Tech)

Department of Civil Engineering

Indian Institute of Technology, Kanpur

INDIA

Thesis Supervisor

Dr. SARVESH CHANDRA

Month and Year of Thesis Submission

August, 1999

CONSOLIDATION SETTLEMENT RESPONSE OF REINFORCED GRANULAR FILL-SOFT SOIL SYSTEM

The present thesis describes a mechanical model for idealizing the settlement response of a geosynthetic reinforced granular fill-soft soil system, by representing

each subsystem with commonly used mechanical elements such as stretched, rough, elastic membrane, Pasternak shear layer, Winkler springs and dashpots.

The model considers several factors governing its behaviour such as the compressibility of granular fill, the compaction of granular fill and the time dependent behaviour of soft soil and prestress in the geosynthetic reinforcement. In this study, the real field situation of three-dimensional consolidation is studied and the results are compared with the studies on one-dimensional consolidation. The response function of the model has been derived for an axi-symmetric condition. The resulting equations in non-dimensional form are solved using an *iterative finite difference* method. The parametric studies carried out, show the effect of various parameters on the settlement response.

ACKNOWLEDGEMENT

I take this opportunity, with pleasure, to pay my respect to my supervisor Dr. Sarvesh Chandra, whose invaluable guidance, constant encouragement on my thought processes, and kind co-operation helped me to able to present this work. It would have been impossible for me to do so without his active involvement in my work.

I am also grateful to the other faculties of Geotechnical Engineering Department, for their valuable advice, suggestion and help, whenever I have asked for.

I must thank Dr. Sanjay Shukla for his sincere help to me.

I will always remember the days I have spent with Anubhav, Ashok, Amol, Sridhar, Kalyan, Sreekanth and Ramu and the happy moments we have shared together.

I thank Sidhu, Antu, Sandy, Sir Bnaru, Deepanjan, Koushik, Chhoru, Probuddha, Sreyangshu, Sudip, Goutamda and Bhuiyanda and all other IEEE members; without whom it would have been impossible to stay at I.I.T.K for the past two years. But, this requires another ninety-page write-up to acknowledge my associations with them, so let me keep it for other occasion.

Anirban

CONTENTS

LIST OF FIGURES

LIST OF TABLES

NOTATIONS

a radius of loaded region

b radius of reinforced zone

c_{vz} the coefficient of consolidation, when the water flow is vertical

c_{vr} the coefficient of consolidation, when the water flow is radial

B half width of foundation

G_t shear modulus of upper shear layer

G_t^* nondimensional shear parameter for upper shear layer ($=G_tH_t/k_sa^2$)

G_b shear modulus of lower shear layer

G_b^* non-dimensional shear parameter for lower shear layer ($=G_bH_b/k_sa^2$)

H_t thickness of upper shear layer

H_b thickness of lower shear layer

H_s thickness of soft soil layer

i space subscript

j time subscript

k_f modulus of subgrade reaction for granular fill

k_s modulus of subgrade reaction for soft foundation soil

k lateral stress ratio

q load intensity

q^* nondimensional load intensity ($=q/k_sa^2$)

q_0 uniform load intensity

q_t vertical stress at the top of membrane

q_b vertical stress at the bottom of membrane

q_b^* nondimensional uniform load intensity ($=q_0/k_sa^2$)

r radius vector of polar co-ordinates

R nondimensional radial distance ($=r/a$)

T mobilized tensile force in membrane per unit length

T^* nondimensional mobilized tensile force per unit length ($=T/k_sa^2$)

T_p pretension in the membrane per unit length

T_p pretension in the membrane per unit length

T_p* nondimensional pretension in the membrane per unit length ($=T_p/k_s a^2$)

T_v time factor

U_z degree of consolidation for the soft foundation soil (one-dimensional consolidation)

U_{zr} degree of consolidation for the soft foundation soil (three-dimensional consolidation)

w vertical surface displacement

W nondimensional vertical surface displacement (=w/a)

X_k dimensionless variable parameters (k=1,2,3,4)

α modulur ratio ($=k_f/k_s$)

θ slope of membrane

μ_t interfacial friction coefficient at the top face of membrane

μ_b interfacial friction coefficient at the bottom face of membrane

τ_{rz} shear stresses on vertical face of the shear layer

ϕ vectorial angle of polar co-ordinates

ϕ' effective angle of shearing resistance

CHAPTER - 1

INTRODUCTION

1.1 GENERAL

A lot of industrialization activity is coming up in the coastal areas, which consist of fine grained or saturated soils, soft in nature. The poor soil conditions in those areas often create problems to Geotechnical engineers associated with the design and construction of foundation and civil engineering structures. Soils with low bearing capacity would hardly withstand the heavier structures. Besides that there are so many existing stable structures resting on soft foundation soil wherein the excessive settlements have adversely affected their functional utility.

Some of the common options to solve the problem are: - excavation and replacement with suitable soil, change of site, designing the structures properly, deep foundations, stabilization with injected activities. One of the popular techniques well suited technically and economically is *ground improvement* technique.

Lot of soil improvement techniques have been developed in past, some of these are soil replacement, preliminary loading, vibroflotation, dynamic consolidation, grouting, thermal stabilization, ground freezing, chemical stabilization, use of stone columns. Among all these existing methods of ground improvement, only ground freezing is applicable to all types of moist soil. Other

methods are depending on the cohesiveness of the soils. Soil reinforcement has become a major part of Geotechnical engineering practice over the last 40 years and it is growing rapidly. The basic principles of this method are simple to understand and have been used by human beings for ages. Rope fibers and bamboo sheets are used to strengthen rural road bases and the foundation soil below low cost buildings.

The modern concepts of soil replacement was proposed by Casagrande who idealized the problems in the form of a weak soil reinforced with high strength membranes laid horizontally in layers (Westergaard, 1938). A French engineer Vidal introduced the modern form of soil reinforcement in the 1960s. His concept was for a composite material formed from flat reinforcing strips laid horizontally in a frictional soil, the interaction between the soil and the reinforcing members being solely by friction generated by gravity. He described this material as 'Reinforced Earth'. The basic attributes of soil reinforcement are reduction in costs and ease of construction. In earlier days reinforced earth technology was mainly involved with steel reinforcement, besides those fiberglasses, nylons, polyesters, and other synthetics in the form of strips, meshes, sheets were used. The most promising new materials belonging to the family of geosynthetic are geotextiles, geogrids and geocomposites, were not regularized before 1973 when a geotextile reinforcement was applied in a bridge construction in Sweden. These are the construction materials with unique properties, which, to a great extent, explain increase in their use in a very short period. In present day geotechnical engineering, the geosynthetics are utilized to serve five major functions; they are separation,

reinforcement, drainage, filtration and moisture barrier. A large number of model tests carried out throughout the world to show that most soils exhibit improved behaviour with respect to their load-settlement response whenever geosynthetic is provided along with granular fill.

The geosynthetic reinforced granular fill-soft soil systems are now being used very frequently as the foundations for shallow footings, unpaved roads, low embankments, heavy industrial equipment etc. Such reinforced soil systems provide improved bearing capacity and reduced settlements by distributing the imposed load over a wider area of weak subsoil. Otherwise a thick granular layer would be needed, which may be very costly and difficult to obtain at the sites. The soil reinforcement is economical, time consuming, and provides ease to construction.

1.2 SCOPE AND ORGANISATION OF THE PRESENT WORK

A critical review of available literature pertaining to the near-surface reinforced foundation soil has been presented in Chapter 2. From the review of literature, it has been observed that there are various aspects such as vertical shear stress transfer at the fill-geosynthetic interface under large deformations, the prestressing of the geosynthetic reinforcement, the compressibility of the granular fill, and the time dependent behaviour resulting from the consolidation of the soft foundation soil, which require their considerations while using the geosyntthetic-reinforced granular fill-soft soil system in the field and estimating its settlement under the applied load.

Chapter 3 deals with the development of a foundation model incorporating different factors, as stated above, which govern the settlement characteristics of geosynthetic-reinforced granular fill-soft soil systems. The model also includes the shear parameter of the granular fill and the characteristics of the soil-geosynthetic interface. Such mechanical modelling approach is being used to soil-structure problems in investigating the gross behaviour and highlighting the major parameters and specialties of the soil without reinforcement.

Further chapter 3 describes the derivation of the response function of the proposed foundation model for axi-symmetric conditions. The finite difference scheme is used for solving the differential equations governing the response of the model. A detailed parametric study is carried out to bring out the effect of each individual parameter on the settlement behaviour.

In chapter 4, the results are presented in a non-dimensional form for practical applications over a wide range of parameters. The present results are compared with those computed using earlier foundation models reported in literature whenever it possible. One of the main features is to make comparison between the behaviour under three-dimensional consolidation case and under one-dimensional consolidation case.

The summary and conclusions of the present work have been presented in chapter 5 along with the recommendations for further work.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

The several theoretical and experimental studies in the area of reinforced soil-foundation interaction have been carried out are being used in structural design of foundation and in the analysis of stresses and deformations within each sub-system of reinforced soil. To explain the reinforcing mechanism of the reinforcement and the assessment of bearing capacity of the soft foundation soil for the estimation of surface deformation under applied load several concepts has been developed. Lot of model tests have been conducted to find the effect of different parameters on the load carrying capacity and the settlement characteristics of the reinforced soft soil system using geosynthetics. Considering different reinforcing mechanisms at the soil-reinforcement interface, lots of numerical model tests have been carried out. This chapter presents a brief review of the state of art on such theoretical and experimental studies associated with reinforced foundation soil under different sections.

2.2 REINFORCING MECHANISM

The relatively inextensible reinforcing materials such as metals, fiber-reinforced plastics etc. having high modulus of deformation can be used and this can be explained using either induced deformation or an induced stress concept.

The induced stress concept related to an apparent cohesion (Schlosser and Vidal, 1969). The tensile strength of the reinforcements and friction at soil-reinforcement interfaces give an apparent cohesion to the reinforced soil system. The friction mobilized at the interface causes a rotation of principle stresses in the soil and modifies the initial state of stresses. Several experimental studies have been carried out in triaxial apparatus on sand samples reinforced by thin aluminum plates (Schlosser and Long, 1972), and by horizontal nets of fiber glass (Yang, 1972) uniformly spaced. Failure occurred, either by breakage of reinforcements or by excessive lateral deformations due to sliding of sand on the reinforcements.

Hausmann (1976) interpreted the effect of reinforcing the sand on its strength characteristics considering a global apparent friction angle (ϕ_R). His assumption is that, the reinforcing effect can be expressed in terms of an increased apparent friction angle when the failure is caused by slip between sand and reinforcements.

The induced deformation concept was presented by Basset and Last (1978). This concept considers the mechanism of tensile reinforcement which involves anisotropy and restraint of the soil deformations in the direction of reinforcements. This effect causes a rotation of the principal directions of the deformation tensor. Analysis of strain fields suggests the ideal reinforcing pattern below a shallow footing. The reinforcement placed horizontally below the footing is the ideal

pattern, i.e.; reinforcements should be placed in the direction of major principal strain.

If geosynthetics are used in the same manner as the inextensible reinforcements, they will also inhibit the development of internal tensile strains in the soil and develop tensile stresses.

It was suggested that the behaviour of the reinforced soil system using extensible reinforcements does not fall within the concepts presented by Vidal (1969) for reinforced soil and therefore, was termed as ply-soil by McGown and Andrawes (1977). The ply-soil, e.g the geosynthetic-reinforced soil has greater extensibility and smaller losses of post peak strength compared to sand alone or reinforced soil. The goesynthetic reinforcement improves the load carrying capacity and settlement characteristics in different ways. The increase of subgrade bearing capacity by changing failure mode, i.e. it tends to force a general failure, instead of a local one. Goesynthetics improve the performance by acting as a separator between the soft soil and the granular fill. This is known as a separation effect of reinforcement.

Nishida and Nishigata (1994) have studied the separation function of geotextiles. Use of geogrids has another benefit owing to the interlocking of the soil through the apertures of the grid membrane known as anchoring effect.

2.3 ANALYTICAL WORKS

Harrison and Gerrard (1972) presented an elastic theory applicable to reinforced earth. In this theory, the reinforced earth was considered to consist of soft soil layers reinforced by closely spaced parallel layers of thin and stiff material. It was pointed out that the suggested method of calculating stresses developed in soft soil and reinforcement can be used for the reinforced earth structures as described by Vidal (1969) for the cases where the stress field is not proportional to depth, e.g., under rafts and footings. By using the methods described, any zone of local overstressing in either the soft soil or the reinforcements can be defined.

Lee and Binquet (1975a) presented an analytical method for designing a reinforced earth slab foundation to carry a particular strip load and defined the term Bearing Capacity Ratio (BCR) as: $BCR=q/q_0$, where, q_0 is the average contact pressure of the foundation on the unreinforced soil; and q is the average contact pressure of the foundation on the reinforced soil, both measured at the same vertical settlement. Based on the observed model test behaviour, three modes of bearing capacity failure were described: (i) ties pullout, (ii) shear failure above uppermost layer of reinforcement, (iii) ties break. For scarcity of definitive data, it was arbitrarily assumed that tie force per layer varies inversely with the number of layers of the foundation. The idealized cost analyses for the for the ultimate designs of a realistic design problem indicated that, if corrosion of the reinforcing material can be neglected, an overall saving up to 100% may be experienced over the cost of

a conventional foundation. However, if corrosion must be considered, the savings will be drastically reduced. It was pointed out that maximum advantages are likely to be found for the short-term construction involving heavy loads over the poor soil conditions.

Giroud and Noiray (1981) developed a method for the design of geosynthetic-reinforced granular fill-soft soil systems used as unpaved roads. The method given considers the reinforcement action of geotextiles alone and does not consider other advantages of geotextiles, such as filtration, separation and drainage. In the suggested method, an allowable rut depth is chosen and making use of a load, spread angle and certain geometric assumptions, the approximate deformed shape of the reinforcement is determined. Assuming that the reinforcement is firmly anchored outside the loaded area, the reinforcement tension and the strain in the reinforcement can be deduced from geometric changes. The reinforcement under the loaded area acts as a curved tensioned membrane, and this results in a higher normal stress on the upper surface of the reinforcement than on the lower surface, such effect was called *membrane effect*. The results were given in the form of charts established using a combination of (i) a quasi-static analysis comparing unpaved roads beheaviour with or without geotextiles, and (ii) formulae relating aggregate thickness and traffic for unpaved roads without geotextiles. This method applies only to purely cohesive subgrade soils and mostly applicable to roads subjected to light to medium traffic.

Miller and Ingold (1982) developed simple theories to model plane-strain compression of a reinforced clay cube, reinforced clay foundation and finally a reinforced clay wall. These are mainly based on the concepts of reinforcing mechanism that the reinforcement embedded in the clay can be assumed to impart an equivalent undrained shear strength to the clay.

Sawicki (1983) analysed reinforced earth considering it as macroscopically anisotropic and homogeneous composite. The suggested rigid-plastic model for reinforced earth was applied for determining the lower bound estimate of the bearing capacity of footing on reinforced earth. It was pointed out that the model does not consider the slippage between the soil and the reinforcement and it has been observed that both the slippage and edge effects are important in the analysis of reinforced earth.

Broms (1987) described a method to stabilize very soft clay using woven geotextile and preloading. By the construction of narrow berms on the geotextile, it is possible to place the fill required for preloading without exceeding the bearing capacity of the soil. The fabric should be stretched as much as possible before the stabilising berms are placed along the perimeter of the geofabric sheet in order to limit the penetration required to build necessary tension in the fabric. It was reported that the method was used in Singapore and Malaysia with satisfactory

results to stabilize very soft clay in settling ponds with shear strength of 3kPa, so that the area could be used for construction. The method suggested that the maximum allowable strain in fabric should be 25% of the ultimate strain for permanent construction and to 50% for temporary structures.

Madhav and Poorooshasb (1988) proposed a 3-parameter mechanical model for geosynthetic-reinforced granular fill-soft soil system. In this model, the geosynthetic reinforcement was represented by a rough elastic membrane. The granular fill and the soft subgrade were idealized by Pasternak shear layer and a layer of Winkler springs respectively. The results at small displacement indicated the effect of granular fill to be more and significant than that of the membrane in reducing the settlements of soft soil system.

Madhav and Poorooshasb (1989) investigated the effect of membrane in increasing the confining stress in the granular material with a consequent increase of the shear modulus (G) with distance in a Pasternak foundation model. Analysis of simple Pasternak foundation with variable G showed a significant reduction in the total settlement as well as in the differential settlements due to the increased shear modulii. It was shown that extending the reinforcement beyond 2B(width of the footing) on either side of the centre of footing has negligible effect on settlements within the loaded region.

Bourdeau (1989) presented a model to assess tensile membrane action in a two-layer soil system reinforced by geotextile. The analysis was based on a two-dimensional plane strain model of the static equilibrium of an elastic membrane placed at the interface between a compressible subgrade and granular base. The theory of probabilistic stress diffusion in a particular medium was used to describe the transmission of load through the gravel base, and a Winkler model was assumed for subgrade. The results showed that the performance of high tensile modulus fabrics was affected to a higher degree by an incomplete anchorage than lower modulus fabrics.

Poorooshasb (1989) developed a procedure for the analysis of geosynthetic-reinforced granular fill-soft soil system by using transform function. The analysis indicates that the contribution from the geosynthetic in supporting the vertical load imposed by foundation was through the tensile stress developed in the grid. The tensile stresses were developed due to the dilatational properties of the fill and the deformation of the system as a whole. As the *overconsolidation ratio* (OCR) for the granular fill (through compaction) increased, the efficiency of the reinforced soil system (defined as the ratio of the load for a predetermined settlement of the geosynthetic reinforced soil to the corresponding value for a soil system in its natural state) increased. It was observed that at lower settlement level (less than 2.5 cm), the presence of geogrids had no effect at all and the efficiency of the system decreased with the increase in the width of footing.

Vokas and Stoll (1989) used a continuum model to describe the response of a horizontally layered elastic system containing one or more reinforcing sheets that may be located at the prescribed depth below the surface. The analysis was based on the well-known equation for layered systems from the linear theory of elasticity. The effect was included by indicating the inter-layer boundary conditions on the basis of an analysis that was similar to that used in the classical theory of thin plates. The results presented for the case of axi-symmetric load represent a limiting case that should be approached by most general models when nonlinear and elastic effects are made small.

Sellmeijer (1990) presented a model for the behavior of a soil-geotextile-aggregate system by combining the membrane action and lateral restraint (slab effect of aggregate). In this model, the aggregate behavior was modelled by the elasto-plastic shear theory, the geotextile by membrane action and lateral restraint, and the subsoil by its bearing capacity. It was pointed out that the model is applicable to low volume, narrow roads to wide parking pools. It is to be noted that this concept of modeling shows much smaller deflections than one, where membrane action alone is considered and is suitable for the design of paved roads.

Madhav and Ghosh (1990) extended the model suggested by Madhav and Poorooshasb (1988) to incorporate the non-linearity of soft soil. It was pointed out

that a single layer of geosynthetic at the interface of the granular fill and the soft soil improves the load response, the improvement being more in case of very soft soil.

Ghosh (1991) further extended the work carried out by Madhav and Ghosh (1990), and Madhav and Poorooshasb (1989) to incorporate the non-linearity of shear stress-shear strain response of granular fill along with the application for multiple layers of geosynthetic reinforcement.

Poorooshasb (1991a) studied the effect of a cavity, which may appear in the subgrade, at some stage after construction of a reinforced fill. This study treated the problem as an equilibrium problem, which is a more realistic representation of the actual case. It was shown that the analysis is not exact, utilizing only a kinematically admissible displacement field.

Poorooshasb (1991b) demonstrated the effectiveness of a transform function in obtaining the solution to the problems of geosynthetic reinforced granular mats on weak subgrades. The types of the problems included are (i) mats bridging over voids appearing in the subgrade after construction, (ii) mats supporting point loads, uniformly or symmetrically distributed loads, (iii) mats placed over nonuniform forms of ground subsidence. The solution given was a kinematic solution, which satisfied most, but not all of the static boundary value problems. The kinetic field employed a fundamental hypothesis, which stated that

all originally vertical material planes remained vertical and did not undergo a change of dimension. The hypothesis was purely based on experimental evidence.

Pichumani (1992) presented an analytical model using the elastic continuum approach to study the interactions between the reinforcements and the soil, and to predict the reduction in surface settlements due to reinforcements below a loaded area, at depth. The mechanism considered was the shear and the normal stress interactions. It was pointed out that the normal stresses resulting from vertical stress interaction for the strips as well as sheets are much higher that the shear stresses resulting from the horizontal stress interactions. The optimum length of strips is 2 to 2.5 times the width of the loaded area.

Dixit and Mandal (1993) applied a variational method to determine the bearing capacity of geosynthetic-reinforced soil. In this method, the shape of the failure surface and the distribution of the normal stress over it were obtained by the use of minimizing theorems of variational calculus. The analysis carried out for determining the bearing capacity of shallow strip foundations loaded vertically and placed on geosynthetic-reinforced sand showed that the shape of the critical rupture surface is log spiral and depends on both cohesion and the angle of internal friction. However, the approach is valid only for shallow foundations.

Espinoza (1994) presented a general expression for evaluating the increase of bearing capacity due to membrane action based on equilibrium conditions. Circular and parabolic shapes were used to simulate the geotextile deformation. It was shown that independent of the model used and geotextile shape assumed, the comparable values for membrane support for relatively small rutting values were obtained. For large rutting ratios, the choice of membrane support model and geotextile deformation shape had significant influence on the results. Models assuming the geotextile deformation to be circular furnished the larger values of the membrane support as compared to the parabolic ones. It was suggested that, under large deformations, model using parabolic fabric deformation, such as Giroud and Noiray's model should be used.

Ghosh and Madhav (1994) extended the work of Madhav and Ghosh (1990) by incorporating the non-linearity of shear stress-shear strain response of granular fill and considering the horizontal shear stress transfer at the fill-geosynthetic surface. This indicated the improvements in the settlement behaviour of the geosynthetic reinforced granular fill-soft soil system subjected to uniformly distributed strip loading. The improvements in the settlement characteristics were significant with respect to stiffness of granular fill, when the soil is softer, and with respect to internal friction, when the fill material is less stiff.

Shukla and Chandra (1994) presented a mechanical model of the geosynthetic-reinforced granular fill-soft soil system; considering all the effects, e.g. the

compressibility, lateral stress ratios of the granular fill, the prestressing of geosynthetic reinforcement and the time dependent behaviour of the foundation model. The have considered the settlement response of model under the one-dimensional consolidation of the soft foundation soil.

Mandal and Joshi (1996) presented a design chart from the limit equilibrium analysis and showed the effect of D/H ratio on the reinforcement required. The reinforcement values required in both the assumptions, tangential and horizontal are very much same. For ($c_u/\gamma H$) values more than 0.25 and for ($c_u/\gamma H$) values more than 0.28, no reinforcement is necessary. The chart gives the required strength of the geosynthetic reinforcement to resist failures.

Zuao (1996) showed the failure load criterion on the geosynthetic reinforced soil structures. The failure criterion of reinforced soil presented is anisotropic due to the inclusion of the geosynthetic reinforcement with preferred direction. The slip-line method in relation with the derived failure criteria can be used for calculating the failure load on the geosynthetic reinforced soil structures. The inclusion of geosynthetic reinforcement enlarges the plastic failure region in reinforced soil structures and significantly increases the load capacity.

2.4 NUMERICAL WORKS

Brown and Poulos (1981) demonstrated the use of finite element model of reinforced earth to increase the bearing capacity and stiffness of foundation soil due to the placement of reinforcement in the soil. Improvement of foundation performance depends on both the number of reinforcing layers and on the concentration of the reinforcement. It was concluded that the quantity of reinforcement require to produce a significant increase in bearing capacity is high.

Andrawes *et al.* (1982) described the finite element method of analysis involving discrete representation of the different constituents within soil-geotextile systems. The method was applied to the prediction of the behaviour of a foundation resting on dense sand with or without a single layer of geotextile placed at different depths in different tests. It was concluded from the measured and predicted data that the influence of geotextile on the load-settlement behaviour of the strip footing is very limited up to settlements equal to approximately 8% of the breadth of the footing.

Love *et al.* (1987) developed a finite element program to tackle the large displacements and strains induced in the physical models of geosynthetic-reinforcement at the fill-soft soil interface. The subgrade was modeled as an elasto-plastic material with limiting shear stress equal to undrained cohesion c_u . The fill

material was modeled as an elastic-frictional material obeying the Matsuoka yield criterion(Matsuoka, 1976).

The reinforcement was treated as perfectly rough so that the failure occurred in soil elements adjacent to the reinforcement rather than the interface.

Koga *et al.* (1988) described the finite element analysis of soil reinforcement system consisting of geogrids used for the two cases, (i) an embankment on soft soil and (ii) a footing. This showed that the use of geogrids as soil reinforcements reduced the tensile stresses in the weak subsoil for the case of embankments and did not influence the settlement characteristics of the embankment. However, the maximum settlement was considerably reduced both in the case of surface and embedded foundations. It was concluded that the geogrids are better than strips as soil reinforcements.

Wu *et al.* (1992) carried out finite element analysis to investigate the effectiveness of using geosynthetic tensile reinforcements for strengthening two highways test embankments, 8.5m and 14,6m height, constructed over weak and highly pervious foundations. The results indicated that the use of tensile reinforcement near the base of the embankments, which were constructed on weak and pervious foundations, had little effect on reducing vertical settlements of the embankments

Otani *et al.* (1994) carried out the bearing capacity analysis of geogrid reinforced foundation ground using rigid plastic finite element method. In order to take into account the reinforcing effect in the analysis, a composite type model including geogrid and the sorrounding soil was proposed. It was concluded that the bearing capacity of geogrid foundation ground increased as the depth and length of the reinforcement were increased, but there existed an optimum depth in order to mobilize the maximum reinforcing effect.

2.5 EXPERIMENTAL WORKS

Binquet and Lee (1975b) carried out tests for about 65-model strip footing on sand, reinforced with the strips of aluminum foil for bearing capacity. The results showed that the load settlement and the ultimate bearing capacities of the foundation could be improved by a factor about 2-4 times above the same load settlement or bearing capacity of an unreinforced soil for otherwise identical conditions. The bearing capacity continued to improve with increasing number of layers up to at least 6 to 8. Tie pull out failure generally occurred with lightly reinforced slabs, $N<2$ or 3, whereas tie breaking, which occurred in the uppermost layers, was associated with heavily reinforced slabs, $N>4$.

Akinmusuru and Akinbolade (1981) conducted laboratory-scale bearing capacity model tests on square footing on a homogenous sand bed reinforced with strips of a local rope material. Results showed that the bearing capacity of footing

depended on the horizontal spacing between strips, vertical spacing between layers, depth below the footing of the first layer, and the number of layers of reinforcement. It was shown that depending on the strip arrangement, ultimate bearing capacity values could be improved by a factor of up to three to four times that of the unreinforced soil.

Sowers *et al.* (1982) investigated the mechanism of failure of aggregate surfaced roads on a very weak subgrades by (i) study of selected road failures, (ii) large scale static load tests, (iii) small scale load tests and (iv) full scale, moving vehicle loading. It was observed that geotextile provided tensile restraint for the aggregate, which increased the load spreading to the subgrade. This reduced elastic deflection with a light load causing failure after two load repetitions.

Fragaszy and Lawton (1984) conducted series of laboratory model tests designed to determine the influence of soil density for a wide range of relative densities (Dr =51%-90%) the reinforcing strip length on the load-settlement behaviour of reinforced sand. Failure of rectangular footings on dense reinforced sand occurred at a larger settlement than an identical footing on unreinforced sand at the same density. As strip length increased from 3 to 7 times the footing width the bearing capacity ratio also increased rapidly.

Dembicki *et al.* (1986) conducted model tests of rigid strip foundation on subsoil reinforced by horizontally placed geotextile. It was pointed out that the

effect of reinforcement was observed at big deformations only, with maximum influence on settlement s, at B/2<s<B (B=foundation width).

Love *et al.* (1987) demonstrated the effectiveness of geogrid reinforcement, placed at the base of a layer of granular fill on the surface of soft clay by small scale model tests. In the tests, monotonic loading was applied by a rigid footing under plain strain conditions, to the surface of reinforced and unreinforced systems, using a range of fill thickness and subgrade strengths. It was observed that the membrane action of the reinforcement becomes significant at large deformations only.

Sakti and Das (1987) investigated the ultimate bearing capacity of a model strip foundation resting on soft clay internally reinforced with geotextile layers in the laboratory. It was found that the geotextile layers placed under a foundation within depth equal to the width of the foundation had some influence on the increase of short-term ultimate bearing capacity. It was pointed out that the minimum length of the reinforcing geotextile layers for maximum efficiency was about four times the width of the foundation.

Kim and Cho (1988) investigated the effects of geotextile reinforcement on bearing capacity and the deformation of soil foundation in view of the distance of footing from geotextile layer and the footing embedment ratio. It was pointed out that the ratio of sand layer depth on soft clay layer of strip footing width, which gave the beneficial effects of geotextile, fell between 0.5 and 1.0 for the settlements non-dimensionalised with width of the footing were less than 1.0.

Das (1989) prsented laboratory model tests results for ultimate bearing capacity of strip and square shallow foundations supported by a compact sand layer underlain by a soft clay with and without geotextile at the sand-clay interface. The results showed that with the use of geotextile, the critical value of H/B ratio at which the maximum bearing capacity ratio occurred was about 0.75 for strip foundations and about 0.5 for square foundations, where B is the width of foundation, and H is the thickness of compacted sand layer below the base of the foundation. These values of H/B were about half of those obtained without geotextiles.

McGown *et al.* (1990) conducted an extensive program of tests for monotonic loading on footings incorporating bamboo rods, which possessed both bending and tensile stiffness, together with a layer of geotextile at the sand-clay interface. Results showed that large increase in bearing capacity might be achieved even at low deformations.

Mandal and Sah (1993) carried out bearing capacity tests on model footings on clay subgrades reinforced with geogrids placed horizontally. Test results showed that the effectiveness of geogrid reinforcement increased the bearing capacity of clay subgrades, with improvements being observed at nearly all levels of deformations. It indicated that the maximum percentage reduction in settlement with the use of geogrid reinforcement bellow the compacted and saturated clay was

about 45% and it occurred at a distance of 0.25B (B=width of the footing) from the base of the square foundation.

Omar *et al.* (1993) carried out laboratory model test for the ultimate bearing capacity of strip and square foundation supported by sand reinforced with geogrid layers. Based on the test results, the critical depth of the reinforcement and the dimensions of geogrids layers for mobilising the maximum bearing capacity ratio were determined and compared. For the development of maximum bearing capacity, the effective depth of reinforcement was about 2B for strip foundations where, B was the width of the strip foundation and 1.4B for square foundations where B was the length of each side of the square.

Puri *et al.* (1993) conducted number of laboratory model tests on a square surface foundation supported by sand with and without critical geogrid reinforcement, A cyclic load was superimposed on the initial static load, and the permanent settlement of the foundation during cyclic load was monitored. Ultimate permanent settlement was severely reduced due to geogrid reinforcement.

Das and Shin (1994) carried out laboratory model tests to determine the permanent settlement of a surface strip foundation supported by geogrid-reinforced saturated clay and subjected to a low frequency cyclic load. It was concluded that for a given amplitude of the cyclic load intensity, the maximum permanent settlement increased with the increase of the intensity of the static load and for a

given intensity of static loading, the maximum permanent settlement increased with the increase in the amplitude of cyclic load intensity.

Floss and Gold (1994) examined the improvement of the bearing and deformation behaviour by means of a geosynthetic reinforcement placed at the interface of granular fill and soft clay. It was pointed out that with reinforcement the granular layer was able to transmit shear forces on a higher level without collapsing.

Khing *et al.* (1994) presented laboratory model tests results for ultimate bearing capacity of a surface strip foundation supported by a strong sand layer of limited thickness underlain by a weak clay with a layer of geogrid at the sand-clay interface. Based on the results, it appeared that the optimum height of the strong sand layer should be about two-third that of the foundation width for obtaining the maximum benefit from the geogrid reinforcement in increasing the ultimate bearing capacity.

Manjunath and Dewaiikar, (1994) carried out model footing tests to obtain the effects of a single layer of geosynthetic reinforcement on the bearing capacity of shallow foundations. It was pointed out that the primary properties of the reinforcement materials that affect the performance of footings on reinforced soil beds were their tensile strength, elastic modulus and aperture size.

Nishida and Nishigata (1994) carried out laboratory tests by applying cyclic load on the surface of the pavement model in cylindrical mould to evaluate the separation function of the geotextile and to find out the relationship. It was pointed out that, the reinforcement was a prime function, when the ratio of the applied stresses on the subgrade soil to the shear strength of the subgrade soil (σ/c_u) was high. However, the separation could be an important function when the ratio is low.

Yetimoglu *et al.* (1994) investigated the bearing capacity of rectangular footings on geogrid-reinforced sand by performing laboratory model tests as well as finite element analyses. The analysis indicated that the increasing reinforcement stiffness beyond 1000kN/m would not bring about any further increase in the bearing capacity.

Athanasopoulos (1996) made direct shear test on the near-saturated silty clay samples reinforced with woven and non-woven geotextiles. Those were presented and analyzed in terms of the strength increase, shear and volumetric deformation of reinforced soil and soil/reinforcement interface bond development. Analyses based on the total stresses, indicated that the inclusion of non-woven geotextiles resulted in significant strength increase of wet cohesive soil. The inclusion of woven geotextiles, however, did not offer any strength increase.

Das *et al.* (1998) presented model laboratory tests results for square surface foundation supported by geogrid reinforced sand and subjected to transient loading.

The tests were conducted with one model foundation at one relative density of compaction using only one type of geogrid. Based on the model test results, it appeared that the geogrid reinforcement reduced the settlement of foundation. The settlement reduction factor is a function of the depth of the reinforcement.

2.6 CONCLUSIONS

In the previous sections, a critical review of the available literature pertaining to the near-surface reinforced foundation soil is given. From the results of a large number of model tests conducted till very recently and also from the results presented through the various analytical and numerical studies of the geosynthetic-reinforced granular fill-soft soil system, it is observed that the geosynthetics, particularly geotextiles and geogrids show their beneficial effects as reinforcements only after large settlements (Andrawes *et al.*, 1982; Guido *et al.*, 1985; Rowe and Soderman, 1987; Madhav and Poorooshasb, 1988; Poorooshasb, 1989) which may not be a desirable feature for shallow foundations, paved and unpaved roads, embankments etc.

Prestressing the geosynthetic reinforcement can be one technique, which can reduce the settlements of the geosynthetic-reinforced foundation soil significantly.

The idea of prestressing has been recognized in the past (Barashov *et al.*, 1977; Aboshi, 1984; Watary, 1984; Broms 1987; Hausmann, 1990; Koerner, 1990). In

fact most of the earlier workers have realized the importance of prestressing the geosynthetics but analytical work is scarce.

It is seen that most of the existing simple foundation models, especially mechanical models, consider only the horizontal shear transfer mechanism at the soil-geosynthetic interface and hence their application are limited to problems involving infinitesimal deformations only. For the inclusion of large deformations, both the horizontal and vertical shear transfer mechanisms should be considered.

It is also seen from the literature review that the compressibility of the granular fill has been neglected in the most cases, which might have been important effect on the settlement behaviour, especially where a layer of loose compressible granular fill used. It is also observed that the simple model for the estimating the settlements of the geosynthetic-reinforced granular fill-soft soil system at different stages of consolidation of the soft soil has not yet been developed.

Shukla and Chandra (1994) considered some of the effects stated above and developed a foundation model to show the effects of various parameters on the geosynthetic-reinforced granular fill-soft soil system. However, they considered the case of one-dimensional consolidation only, which is not very realistic in most of the field problems.

So, there is a need of development of foundation model to incorporate all those parameters and also the case of three dimensional consolidation in a simple way to estimate the settlements by considering most of the factors governing the behaviour under specific field situation.

CHAPTER 3

DEVELOPMENT OF A FOUNDATION MODEL

3.1 GENERAL

One of the common approaches to solve many geotechnical engineering problems involving interaction between the structural foundation and supporting soil subgrade is to idealize the soil subgrade by mechanical foundation models, such as Winkler model, Pasternak model, Filonenko-Borodich model, Kerr model etc. The models idealized the behaviour of one soil layer only. The concepts involved in these models are general in nature and can be extended to idealize the behaviour of two soil layers and also to reinforced granular beds resting on soft foundation soils.

The models for general soil behaviour as described above need some modifications before the same may be used for idealizing the characteristics of geosynthetic-reinforced granular fill-soft soil system. The geosynthetic reinforcement introduced into the soil works through lot of mechanisms such as membrane effect, confinement effect, separation effect and anchoring effect. In the present chapter, a mechanical foundation model for geosynthetic-reinforced granular fill-soft soil system is developed to study the settlement response of the system.

In this chapter, a mechanical foundation model for geosynthetic-reinforced granular fill-soft soil system is developed to incorporate the effect of three-dimensional consolidation in the settlement response of the system. This is an improvement over the model proposed by Shukla, 1995. The main factors affecting the response of the model are horizontal and vertical shear stress transfer mechanism, prestressing of the

geosynthetic reinforcement, compressibility of the granular fill and the time dependent behaviour of the soft foundation soil. The numerical solutions are obtained by an *iterative finite difference* scheme and the results are presented in a non-dimensional form.

3.2 RESPONSE FUNCTION OF THE MODEL

In particular applications, the geosynthetic-reinforced granular fill-soft soil system are often used as foundations for parking lots, warehouses, tank filled with liquid and columns, circular in plan. The analyses of such foundations are attempted by taking the problem as an axi-symmetric one. Considering the equilibrium of forces on different elements of the reinforced soil system at any instant derives the equations governing the response of the proposed model at that particular instant (t>0).

Figure 3.1(a) shows a geosynthetic-reinforced granular fill-soft soil system subjected to axisymmetric loading. The characteristics of such a system is idealized by the proposed foundation model as shown in Fig 3.1(b) along with the system of co-ordinates considered in the derivation of governing equations of the response model. The vertical force equilibrium equations of the upper and lower shear layer elements as time t>0 (Fig. 3.2(b) &(c)), can be written as:

$$q = q_t - G_t H_t \left(\frac{\partial^2 w}{\partial r^2} + \frac{1}{r} \frac{\partial w}{\partial r} \right) \quad (3.1)$$

and

$$q_b = q_s - G_b H_b \left(\frac{\partial^2 w}{\partial r^2} + \frac{1}{r} \frac{\partial w}{\partial r} \right) \quad (3.2)$$

where, r is the distance measured from the centre of the axi-symmetrically loaded region along radial direction.

The expression q_s can be written as:

$$q_s = \frac{\alpha k_s w}{1+\alpha U} \quad (3.3)$$

where, $\alpha = \frac{k_f}{k_s}$

and U is the degree of consolidation given as:

$$U = 1 - \frac{u_e}{u_0} \quad (3.4)$$

u_e is the average excess pore water pressure at time t and u_0 is the initial excess pore water pressure. In this present model, this u_e is solved with the help of finite difference technique.

3.2.1 CONSOLIDATION EQUATIONS

3.2.1.1 One Dimensional Consolidation

Shukla (1995) obtained u_e by using one-dimensional consolidation and the u_e is expressed by the following equation:

$$u_e = \sum_{m=0}^{m=\infty} \frac{2u_0}{M^2} e^{-M^2 T_v} \quad (3.5)$$

where, $M=(2m+1)\pi/2$, $m=0,1,2.....$; $T_v(=c_{vz}t/H_s^2)$ is the time factor for primary consolidation, c_{vz} is the coefficient of consolidation, when the pore water pressure dissipation is vertical only, H_s is the thickness of soft foundation soil. The equation (3.5) is the well known analytical solution of the Terzaghi consolidation equation for saturated soft foundation soil with the boundaries as shown in Fig. 3.1 and for a uniform initial pore water pressure distribution (Terzaghi, 1943). The other way to obtain the value of average excess pore water pressure u_e is the solution of a governing differential equation, which is given below,

$$c_{vz}\frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} \qquad (3.6)$$

The solution of the equation (3.6) gives the value of u_e at the particular time $t>0$.

3.2.1.1a Boundary Conditions

At the base of the footing level (z=0), no flow is there, $(\frac{\partial u}{\partial z} = 0)$ and at the boundary along the vertical flow ($z=H_s$), pore water pressure is zero.

3.2.1.2 Three Dimensional Consolidation

In the present study, the governing differential equation for an axi-symmetric case of three-dimensional consolidation is given as,

$$c_{vz}\frac{\partial^2 u}{\partial z^2} + c_{vr}\left(\frac{\partial^2 u}{\partial r^2} + \frac{1}{r}\frac{\partial u}{\partial r}\right) = \frac{\partial u}{\partial t} \qquad (3.7)$$

where, c_{vr} is the coefficient of the consolidation, when the dissipation of pore water pressure is considered to be in the radial direction. Thus in the equation (3.7), the pore

water pressure dissipation is considered to be in all directions. The soil is homogenous and isotropic, and it may be assumed that $c_{vz}=c_{vr}=c$.

Then, the three-dimensional consolidation equation simply reduces to,

$$\frac{\partial^2 u}{\partial z^2}+\frac{\partial^2 u}{\partial r^2}+\frac{1}{r}\frac{\partial u}{\partial r}=\frac{1}{c}\frac{\partial u}{\partial t} \qquad (3.8)$$

To obtain the solution of the equation (3.8), the whole soft foundation soil layer is divided into 400 elements.

The finite difference form of the equation can be written as,

$$\left(\frac{u_{i,j+1,k}-2u_{i,j,k}+u_{i,j-1,k}}{(\Delta z)^2}\right)+\left(\frac{u_{i+1,j,k}-2u_{i,j,k}+u_{i-1,j,k}}{(\Delta r)^2}+\frac{1}{r}\frac{u_{i+1,j,k}-u_{i-1,j,k}}{2\Delta r}\right)=\frac{1}{\mathrm{c}}\frac{u_{i,j,k+1}-u_{i,j,k}}{\Delta t} \qquad \ldots \quad (3.9)$$

Where, i and j are space subscripts and k is the time subscript; i is taken along the radial direction and j is taken along the vertical direction.

3.2.1.2a Boundary Conditions

Since as an axi-symmetric system is being considered, only half width of the foundation (B) is studied.

(i) At the centre of the reinforced zone (r=0), no flow is there ($\frac{\partial u}{\partial r}=0$); and at the boundary along the radial flow (r=B) pore water pressure is zero(u=0).

(ii) At the base of the footing level, (z=0), no flow is there ($\frac{\partial u}{\partial z}=0$); and at the boundary along the vertical flow ($z=H_s$) pore water pressure is zero.

3.2.1b Method of Solution

To solve the equation (3.9), $\frac{\partial^2 u}{\partial z^2}, \frac{\partial^2 u}{\partial r^2}$ and $\frac{\partial u}{\partial r}$ are expressed by central difference scheme and $\frac{\partial u}{\partial t}$ has been expressed by forward difference scheme. In this study both the equations are solved by using finite difference technique. The equation (3.9) have been solved by taking the same step size both in z and r directions i.e. $\Delta z = \Delta r$. The value of u_e is found out after any particular time t>0 from the equation (3.6) for one-dimensional consolidation case. At that same time t, the average excess pore water pressure u_e is calculated after solving the equation (3.9). After obtaining the nondimensional value u_e this value is substituted in equation (3.4) to easily find out the value of U_{zr} i.e. the degree of three-dimensional consolidation. Thus the comparison of results for both the cases i.e. one-dimensional and three-dimensional consolidation has been made after the same time t. This method is a modified method of Shukla (1995) to obtain the settlement response under the effect of three-dimensional consolidation.

3.2.2 EQUILIBRIUM EQUATIONS

Now, from the Figures 3.1, 3.2 and 3.3 and considering the horizontal and vertical force equilibrium of the rough elastic membrane element at time t>0, one can gets the response function of this proposed model (Shukla, 1995). The governing equations are:

$$q = \overline{X}_1 \frac{\alpha k_s w}{1+\alpha U} - \overline{X}_2 (T_p + T)\cos\theta \frac{\partial^2 w}{\partial r^2} - (G_t H_t + \overline{X}_1 G_b H_b)(\frac{\partial^2 w}{\partial r^2} + \frac{1}{r}\frac{\partial w}{\partial r}) \quad (3.10)$$

and,

$$\frac{\partial T}{\partial r} + \frac{T_p + T}{r} = -\overline{X}_3 \{q + G_t H_t (\frac{\partial^2 w}{\partial r^2} + \frac{1}{r}\frac{\partial w}{\partial r})\} - \overline{X}_4 \{\frac{\alpha k_s w}{1+\alpha U} - G_b H_b (\frac{\partial^2 w}{\partial r^2} + \frac{1}{r}\frac{\partial w}{\partial r})\} \quad (3.11)$$

where, $$\overline{X}_3 = \mu_t \cos\theta (1 + k\tan^2\theta) - (1-k)\sin\theta \quad (3.12a)$$

and, $$\overline{X}_4 = \mu_b \cos\theta (1 + k\tan^2\theta) + (1-k)\sin\theta \quad (3.12b)$$

Equation (3.10) and (3.11) govern the response of the proposed foundation model.

3.2.2.a Method of Solution

$$q^* = \overline{X}_1 \frac{\alpha W}{1+\alpha U} - \overline{X}_2 (T_p^* + T^*)\cos\theta \frac{\partial^2 W}{\partial R^2} - (G_t^* + \overline{X}_1 G_b^*)(\frac{\partial^2 W}{\partial R^2} + \frac{1}{R}\frac{\partial W}{\partial R}) \quad (3.13)$$

and,

$$\frac{\partial T^*}{\partial R} + \frac{T_p^* + T}{R} + \overline{X}_3 \{q^* + G_t^* (\frac{\partial^2 W}{\partial R^2} + \frac{1}{R}\frac{\partial W}{\partial R})\} + \overline{X}_4 \{\frac{\alpha W}{1+\alpha U} - G_b^* (\frac{\partial^2 W}{\partial R^2} + \frac{1}{R}\frac{\partial W}{\partial R})\} \quad (3.14)$$

where, R=r/a, W=w/a, $G_t^* = G_t H_t / k_s a^2$, $G_b^* = G_b H_b / k_s a^2$, $q^* = q/k_s a$, $T_p^* = T_p / k_s a^2$ and $T^* = T/k_s a^2$; 'a' is the radius of the loaded region.

Writing equations (3.13) and (3.14) in finite difference form within the specified time-space domain, for an interior node, (j,k), where j,k are indices for space and time respectively, one gets:

$$q_{j,k}^* = \overline{X}_{1j,k} \frac{\alpha W_{j,k}}{1+\alpha U_k} - \overline{X}_{2j,k} (T_p^* + T_{j,k}^*)\cos\theta_{j,k} \frac{d^2 W}{dR^2}\Big|j,k - (G_t^* + \overline{X}_{1j,k} G_b^*)$$

$$(\frac{d^2W}{dR^2}|j,k + \frac{1}{R_i}\frac{dW}{dR}|j,k)\} \qquad (3.15)$$

and,

$$T^*_{j,k} = \frac{1}{1-\frac{\Delta R}{Rj}}[T^*_{j+1} + T^*_p\frac{\Delta R}{Rj} + (\Delta R/4)[\overline{X}_{3j,k} + \overline{X}_{3j+1,k})\{(q^*_j + q^*_{j+1}) +$$

$$G^*_t((\frac{d^2W}{dR^2} + \frac{1}{R_j}\frac{dW}{dR})|j,k + (\frac{d^2W}{dR^2} + \frac{1}{R}\frac{dW}{dR})|j+1,k)\} + (\overline{X}_{4j,k} + \overline{X}_{4j+1,k}))$$

$$\{\frac{\alpha}{1+\alpha U_k}(W_{j,k} + W_{j+1,k}) - G^*_b((\frac{d^2W}{dR^2} + \frac{1}{R}\frac{dW}{dR})|j,k + (\frac{d^2W}{dR^2} + \frac{1}{R}\frac{dW}{dR})|j+1,k)\}]$$

.... (3.16)

In equation (3.16), the derivatives $\frac{d^2W}{dR^2}$ and $\frac{dW}{dR}$ are expressed by the central difference scheme, while, $\frac{dT^*}{dR}$ has been expressed by forward finite difference scheme. Hence, in order to minimize the numerical error, average values of $\overline{X}_3, \overline{X}_4, q^*, W$ and $(\frac{d^2W}{dR^2} + \frac{1}{R}\frac{dW}{dR})$, for each element, are taken in equation (3.16).

3.2.2.b Loading and boundary conditions

The solutions are obtained for a uniform nondimensional load intensity; q^*_0 acting over a circular region of radius a. Since there is a symmetry about the centre of loaded region, the slope of the settlement profile at the centre of loaded region is taken as zero. At the age of the reinforced zone (at R=b/a, b being the radius of the reinforced zone), the slope is considered as zero as observed in the cases, where the membrane is free or fixed. Mobilized tensile force at the edge of the reinforcement is assumed as zero (T*=0

for R=b/a), which means that the frictional resistance mobilized over the length of the membrane is sufficient to balance the tensile force in it. In field, the non-zero values of mobilized tensile force may occur at the edge of the membrane, if frictional resistance is not sufficient to balance the tensile force.

Based on the above formulation, results are obtained using computer. Due to symmetry of the problem analyzed, only half region of the problem ($R \geq 0$) is considered. The solutions are obtained with the a convergence criterion,

$$\left| \frac{W_i^n - W_i^{n-1}}{W_i^n} \right| \mathrm{X}\ 100\% < \varepsilon_s \qquad (3.17)$$

for all i, where n and n-1 are the present and previous iterations, and ε_s is the specified tolerance, which is considered to be 0.0001 in the present study. The ranges of nondimensional parameter studies are given as Table 3.1.

3.3 CONCLUSIONS

In this chapter, the basic concepts in the development of a mechanical foundation model for geosynthetic-reinforced granular fill-soft soil system are presented. Also the governing equations to solve the settlement response and the method of solution are given in this chapter. The proposed model is effective to estimate the settlement response under the effect of one-dimensional consolidation as well as the three-dimensional consolidation of the soft foundation soil. The differences in the settlement responses are clearly brought out.

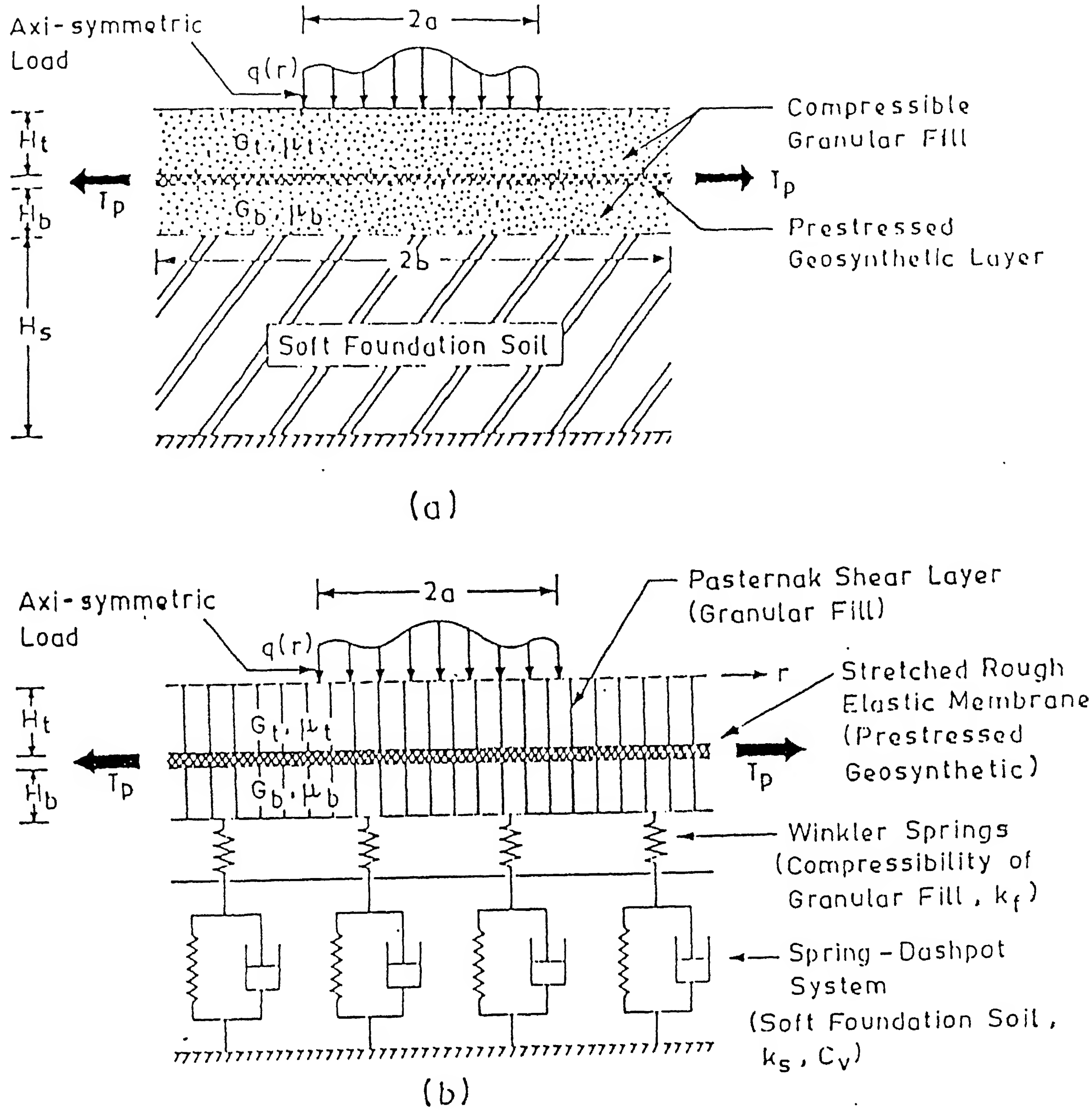

Fig. 3.1. Definition sketch: (a) geosynthetic-reinforced granular fill-soft soil system subjected to axi-symmetric loading; (b) proposed foundation model

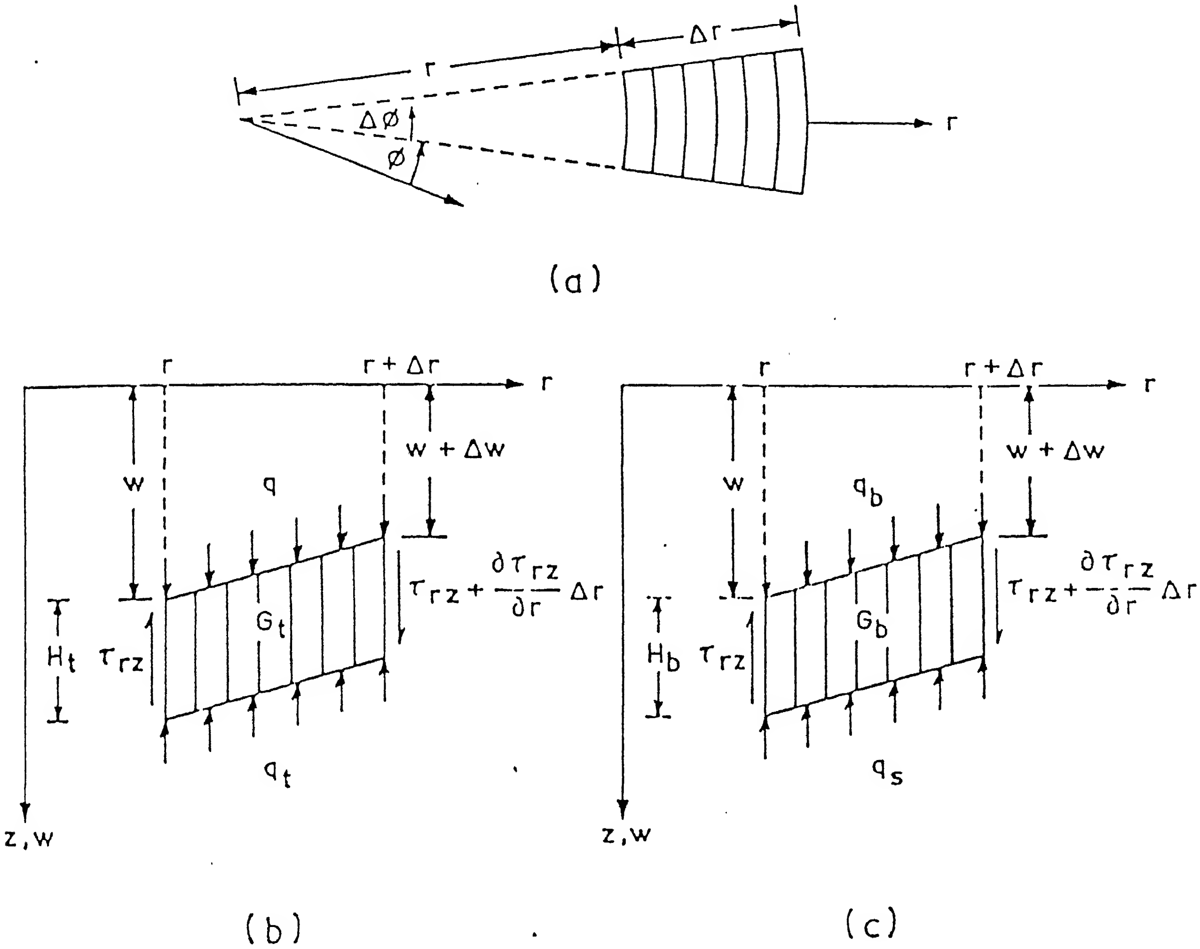

Fig. 3.2. Definition sketch: (a) plan view of shear elements; (b) forces on the upper shear layer element; (c) forces on the lower shear element.

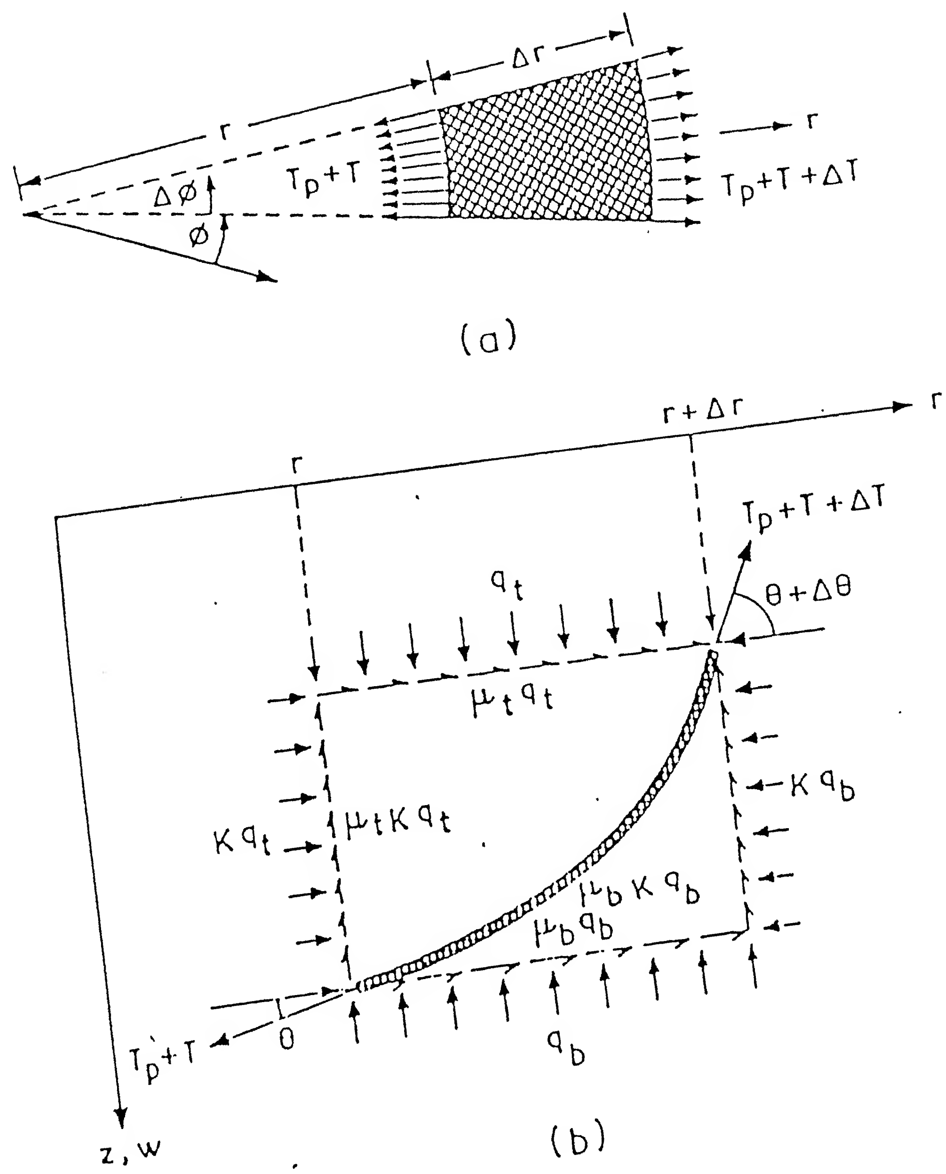

Fig. 3.3. Definition sketch: (a) plan view of the stretched rough elastic membrane; (b) forces on the stretched rough elastic membrane.

TABLE 3.1

Ranges of Nondimensional Parameters studied

Serial Number	Nondimensional Parameters	Ranges
1	Load Intensity	0.01-1.0
2	Shear Parameters, G_t* or G_b*	0.01-1.0
3	Interface friction coefficient, μ_t or μ_b	0.1-1.0
4	Width of reinforced zone, L/B	1.0-3.0
5	Prestress in the geosynthetic reinforcement, T_p*	0.0-1.0
6	Lateral stress ratio, k	0.4-1.0
7	Modular ratio, α (=k_f/k_s)	5-∞
8	Degree of consolidation, U (=U_z, for one-dimensional consolidation, and U_{zr}, for three-dimensional consolidation)	0-100%

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 GENERAL

In this chapter the results obtained by using the model proposed in Chapter 3 are presented in the form of non-dimensional charts. The results obtained are compared to show the effect of one-dimensional consolidation and three-dimensional consolidation of the soft foundation soil, considering the different parameters affecting the settlement response of the geosynthetic-reinforced granular fill-soft soil system. The parameters affecting the response are lateral stress ratios and shear parameters of the granular fill, interfacial friction coefficients of the membrane, prestressing of the geosynthetic reinforcement and compressibility of the granular fill. To study the time dependent behaviour of the soft foundation soil the degrees of three-dimensional consolidation are calculated at the same time and taken as 47%, 70%, 94% instead of 10%, 50% and 90% for one-dimensional consolidation respectively.

4.2 RESULTS

4.2.1 COMPARISON OF RESULTS

Figure 4.1 shows a comparison between the results obtained by Shukla (1995) with the results obtained in the present study by solving the one-dimensional

consolidation equation by finite difference method. For the particular value of the load and the granular fill characteristics, interfacial friction angle and lateral stress ratio for 50 percent consolidation, the results are found to be in good agreement with Shukla's results and the maximum variation is found to be around 5 percent. In this study, the finite difference technique is applied taking different number of elements ranging from 100 to 400. Due to the limitation of the memory, the number of elements was taken as 400. The difference around of 5% has been taken as an acceptable limit.

Figure 4.2 shows the comparison of the results obtained by Shukla (1995) with the results obtained in the present study for three-dimensional consolidation at 70 percent consolidation for different loading for a particular value of lateral stress ratio, shear parameter and interfacial friction. The settlement response is considerably different when three-dimensional consolidation is considered. At smaller load i.e. $q^* = 0.1$, the difference obtained in the settlement is of the order of 40 percent whereas the difference for $q^* = 0.5$ and 1.0 are of the order of 32 and 28 percent respectively. This figure emphasizes the importance of considering three-dimensional consolidation while making a reasonable estimate for the settlement analysis of reinforced granular fill-soft soil system.

4.2.2 THE EFFECT OF THREE-DIMENSIONAL CONSOLIDATION ON DIFFERENT LOADINGS

Figures 4.3 to 4.6 present the load versus settlement response of the granular fill soft soil system for four different loads at different degrees of consolidation for low value of shear parameter. It is observed that the final settlement is always same but

there are significant differences in the settlement response at the various stages of consolidation of the soft soil. The effect of three-dimensional consolidation on the system is much more significant in case of smaller loads. For example, when, q*=0.1, and at 70% consolidation the difference obtained is 36%, whereas, for q*=1.0 and considering all the remaining conditions as same, the difference is 30% at the centre of the loaded region. At the edge no significant change has been found.

Figures 4.7 to 4.10 present the load versus settlement response of the granular fill-soft soil system for four different loads at different degrees of consolidation for high value of shear parameters. Considerable difference in the settlement at both the locations, i.e. centre of the loaded region and the edge has been observed. So, the consideration of three-dimensional consolidation is very important in this study. The difference in settlements at the centre of the loaded region is much higher at smaller loads but it is alomost in the same order at the edge of the loaded region. For example, at q*=0.1; the difference in the settlement is obtained as 30% at centre and 55% at the edge; whereas, for q*=1.0, those are 20% and 52% respectively. In general, the difference in settlement obtained is much higher at the edge of the loaded region in this kind of stiffer soils. So, the effect of three-dimensional consolidation is necessary in this study and it plays a significant role.

Figures 4.11 to 4.14 present the load versus settlement response of the granular fill-soft soil system for four different loads at different degrees of consolidation, considering the typical values of all the parameters and no prestress force. In this case, the difference in settlement obtained under the effect of one-dimensional and three-dimensional consolidation is much higher at smaller loads and it doesn't affect the

edge of the loaded region at all. For example, at 70% consolidation the difference obtained at the centre of the loaded region are 45% and 26% at q*=0.1 and 1.0 respectively.

Figures 4.15 to 4.18 present the settlement response of the system at different degrees of consolidation and various loads under the effect of prerstressing the geosynthetic reinforcement. The significant differences in the response are observed at the centre of the loaded region as well as at the edge also. The differences are higher at smaller loads. For example, at q*=0.1 and for 70% consolidation, the difference observed are 37% and 60% at the centre and at the edge of the loaded region respectively. For higher load of q*=1.0 and at the same instant; those differences are 32% and 40% respectively. The settlement observed in the case of three-dimensional consolidation is always much higher than the case of one-dimensional one at the initial stage of the process.

Figures 4.19 to 4.22 present the load versus settlement response of the geosynthetic-reinforced granular fill for four different loads for different degree of consolidation at the low interfacial friction at the face of the membrane. The difference is as usual much higher in case of smaller loads and at the initial stage of the consolidation process. There is no significant effect on the settlement response at the edge of the loaded region. For example, at q*=0.1 and 70% consolidation, the difference is of the order of 50%, whereas, at higher load, i.e. q*=1.0 and at the same instant, the difference obtained is 40% at the centre of the loaded region.

Figures 4.23 to 4.26 present the load-settlement response of the system for different degrees of consolidation, but at higher interfacial friction value, i.e. $\mu_t=\mu_b=1.0$. In this

case the difference in settlement response for the different degrees of consolidation are much less than the case of low friction values. But, the difference obtained is having similar trend i.e. much higher difference at smaller loads and at the initial stages. For example, at 70% consolidation, the differences are 45% and 32% at q*=0.1 and 1.0 respectively. At 47% consolidation, the corresponding values are 320% and 300% respectively.

Figures 4.27 to 4.30 present the load-settlement response for the four different loads under different degree of consolidation and considering, the low lateral stress of the granular fill. The trends of enhancement of the settlement response at the initial stages of consolidation are clearly observed. The differences in settlement at different degrees of consolidation are much more significant at the smaller loads. For example, at q*=0.1 and 94% consolidation, the difference observed is 5%, whereas for higher loads, i.e. q*=1.0, the difference is 3.5%. The corresponding values at 70% consolidation are 42% and 32% respectively. These differences are even much more higher in the case of lower degree of consolidation, i.e. about 300% at 47% consolidation.

Figures 4.31 to 4.34 present the load-settlement response for different degree of consolidation and for four different loadings at higher value of lateral stress ratio of the granular fill of the reinforced fill-soft soil system, i.e. k=0.7. The differences in settlement response are almost similar as in the case of low lateral stress values. The changes in the settlement response under different degree of consolidation are much higher at smaller loads. For example, at 70% consolidation the difference in settlement responses are 41% and 27% at q*=0.1 and 1.0 respectively. The corresponding values

are 340% and 270% in the case of 47% consolidation. Thus, the three-dimensional consolidation plays an important role in the settlement response at reinforced granular fill-soft soil foundation. No significant changes at the edge of the loaded region have been observed.

Figures 4.35 to 4.38 present the settlement response for four different loadings, considering the different degrees of consolidation at the low value of modular ratio of the granular fill, i.e. α=5. The difference obtained at the initial stages and final stages are not so significant. That difference is almost similar at higher and lower value of loads. For example, for 70% consolidation those values are 22% and 17% at q^*=0.1 and q^*=1.0 respectively. Those differences are 60% and 40% for 47% consolidation.

Figures 4.39 to 4.42 present the settlement response for four different loadings at different degrees of consolidation for higher value of modular ratio, i.e.α=50. The settlement response is following the general trend of behaviour. At smaller loads and at the initial stages, the difference is higher. For example, at 70% consolidation and q^*=0.1, the difference in settlement response between one-dimensional and three-dimensional consolidation is 35%, corresponding value is 25% for q^*=1.0. Those values are 230% and 165% at 47% consolidation and for q^*=0.1 and 1.0 respectively.

4.3 CONCLUSIONS

The proposed foundation model has been employed successfully for the study of the consolidation settlement response at different loadings of geosynthetic reinforced granular fill-soft soil system. The effects of different parameters on the model have similar trend as observed in the chapter 3.

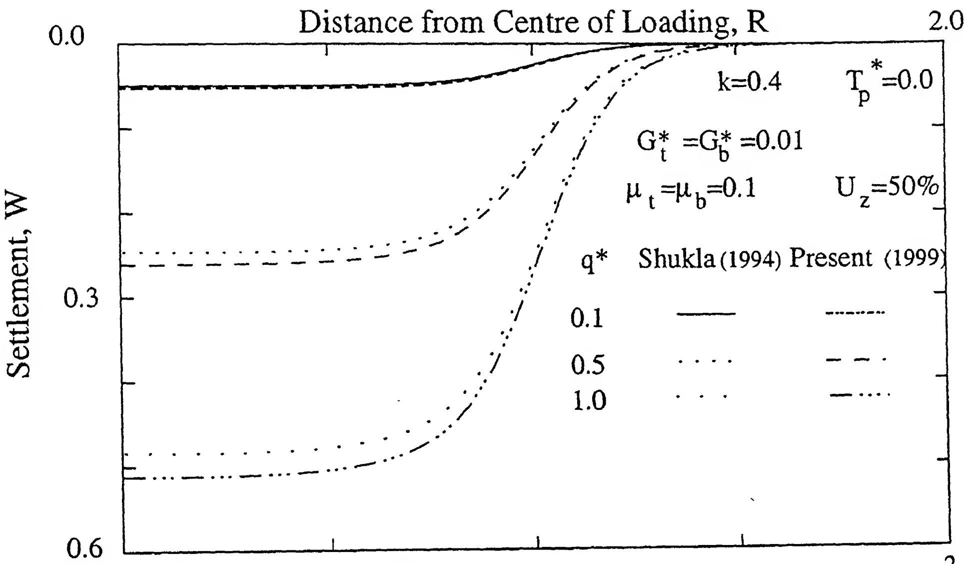

Fig. 4.1 Load-settlement profiles-comparison of models

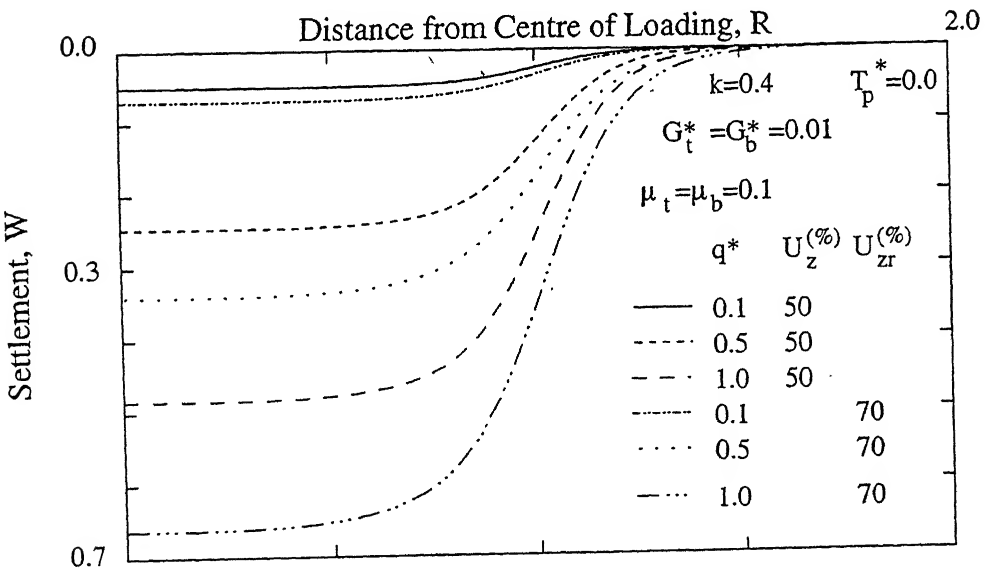

Fig. 4.2. Consolidation settlement at different loadings

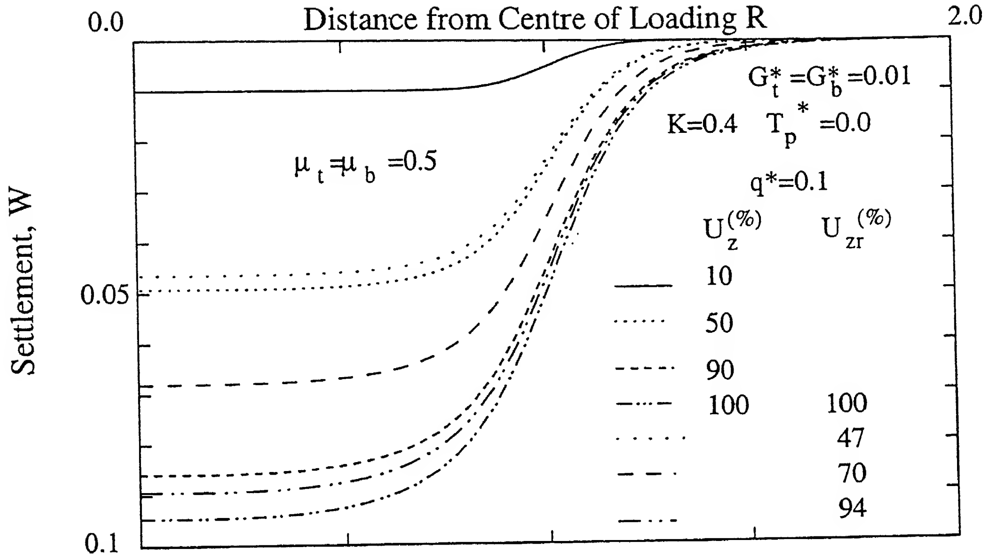

Fig. 4.3. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.1)

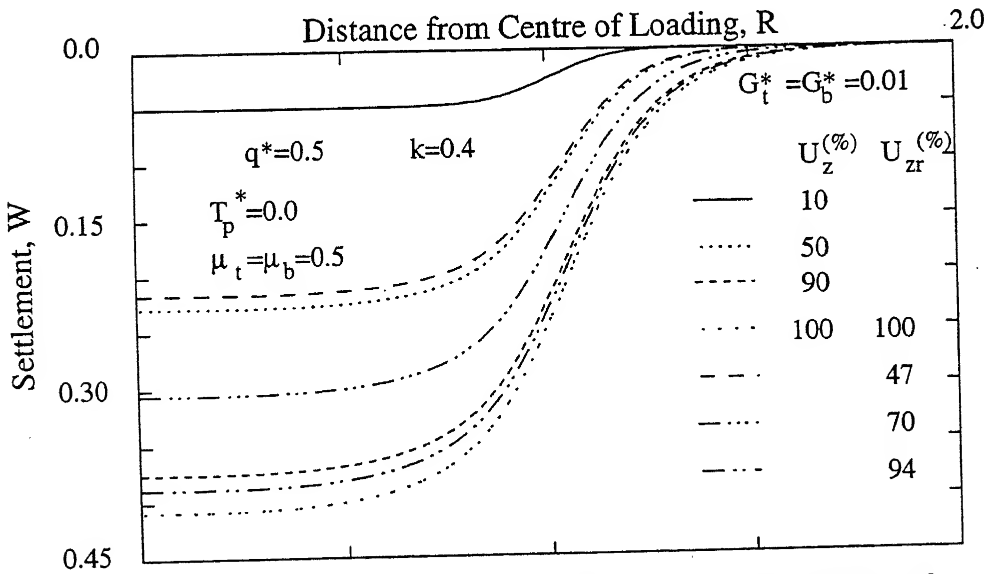

Fig. 4.4. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.5)

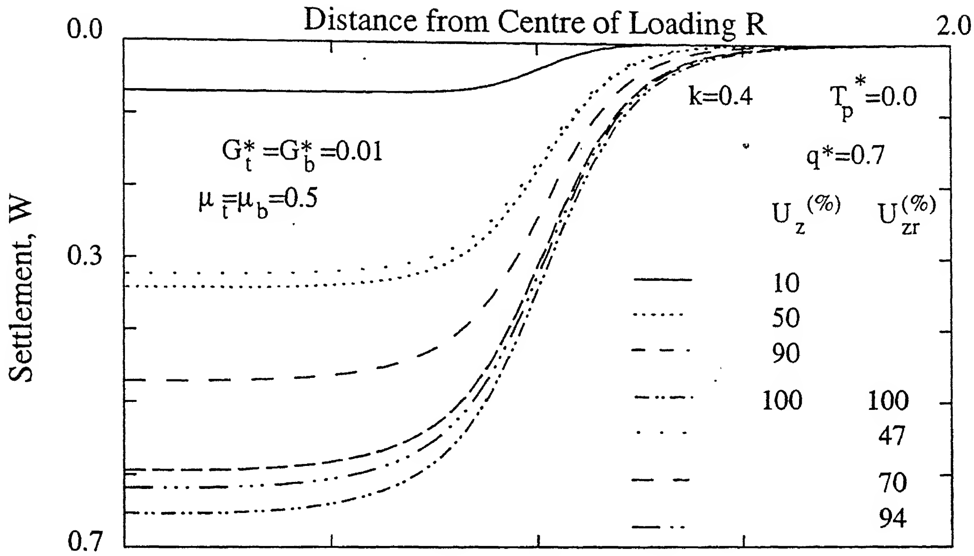

Fig. 4.5. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.7)

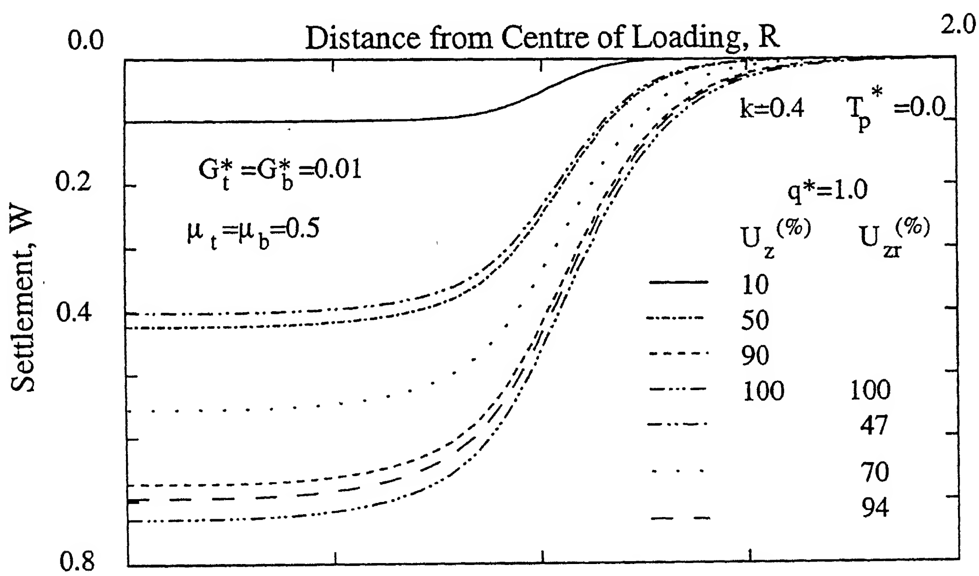

Fig. 4.6. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=1.0)

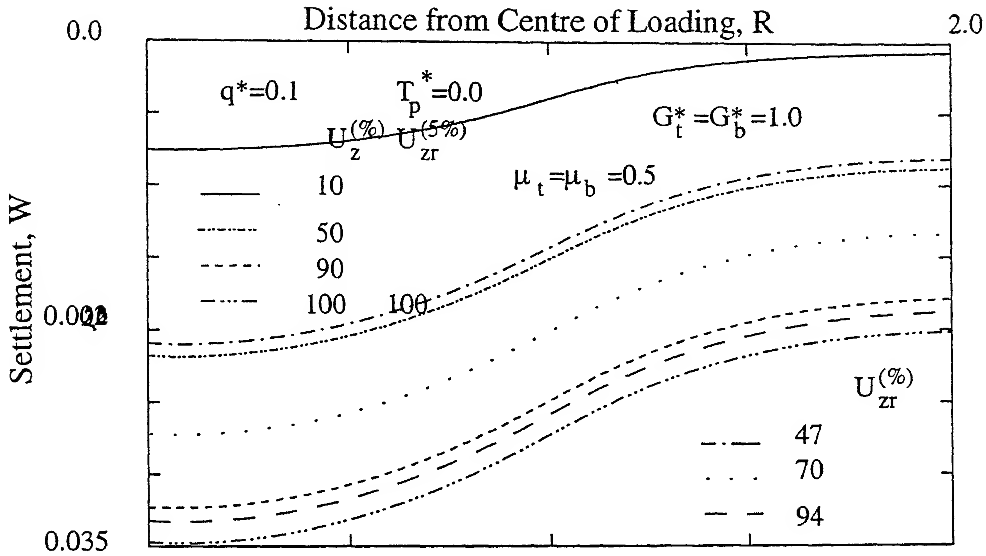

Fig 4.7. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.1)

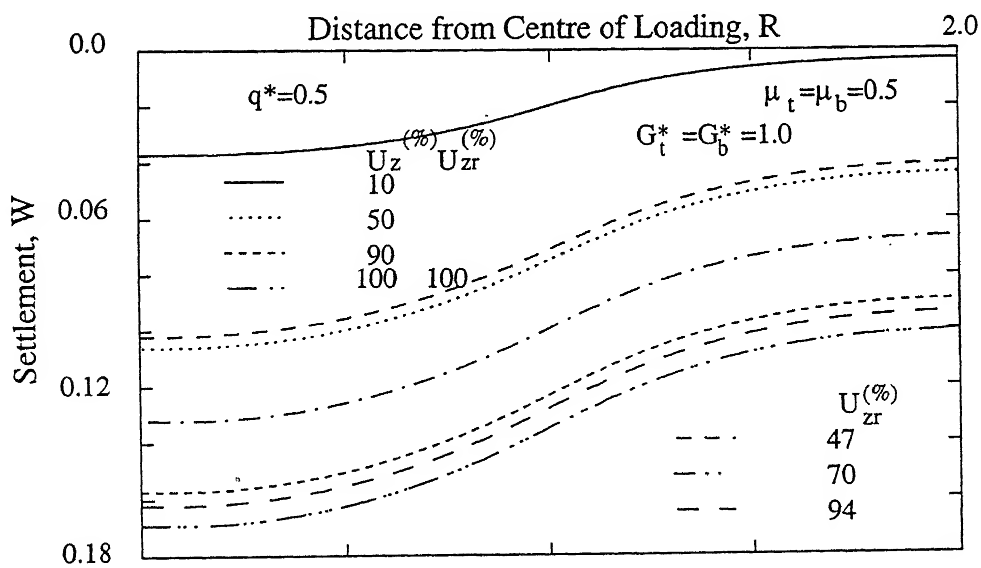

Fig 4.8. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.5)

CENTRAL LIBRARY
I. I. T., KANPUR
No. A 130459

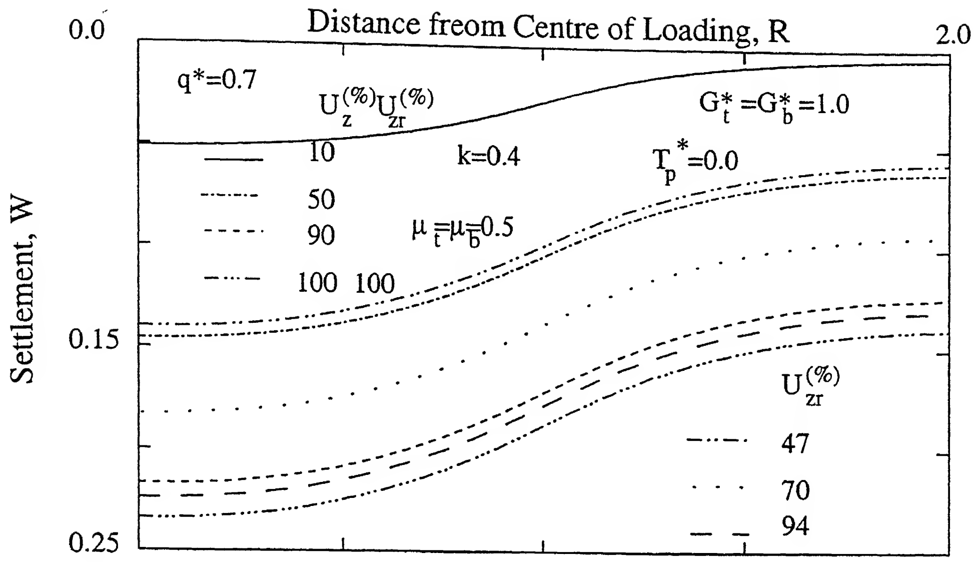

Fig. 4.9. Load-settlement profiles at various stages of consoildation of the soft foundation soil (for, q*=0.7)

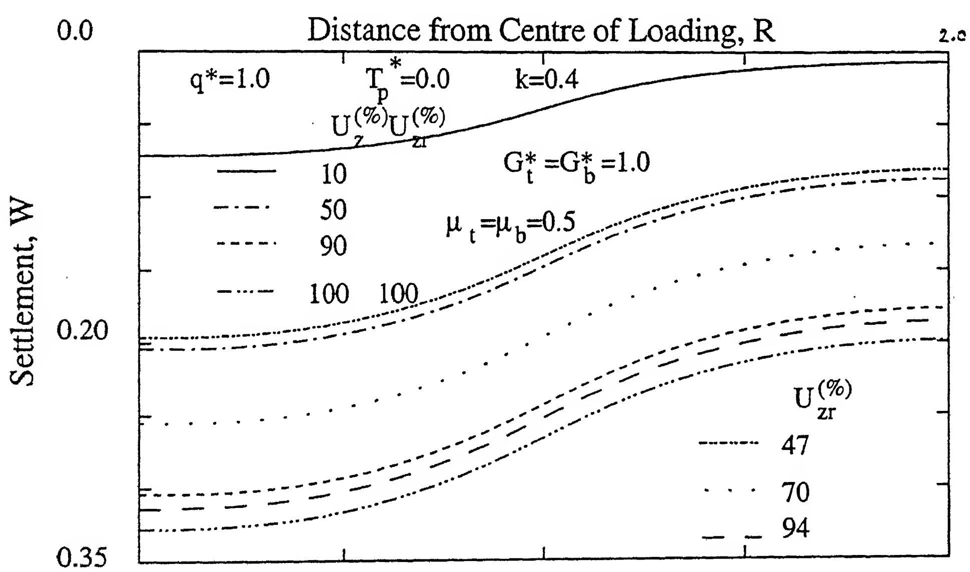

Fig. 4.10. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=1.0)

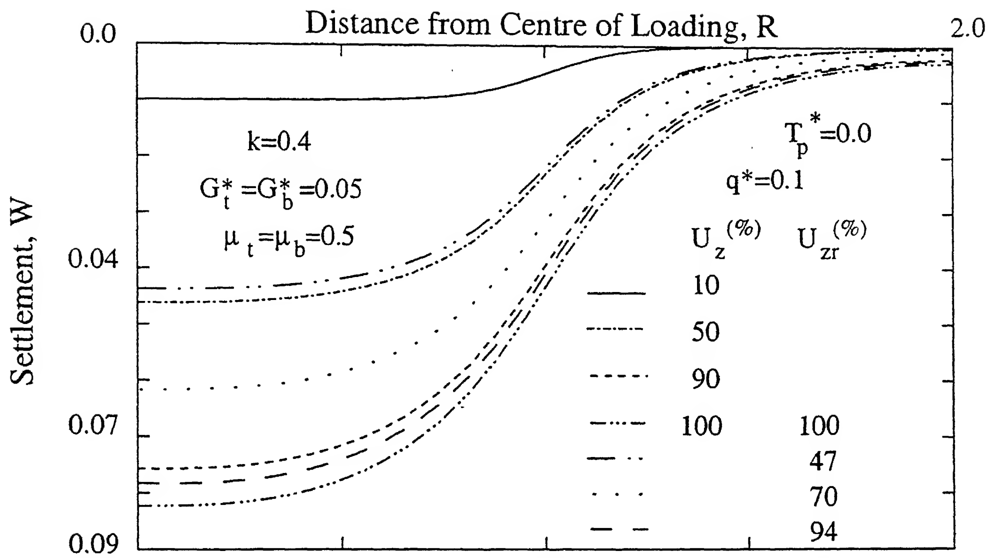

Fig. 4.11. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.1)

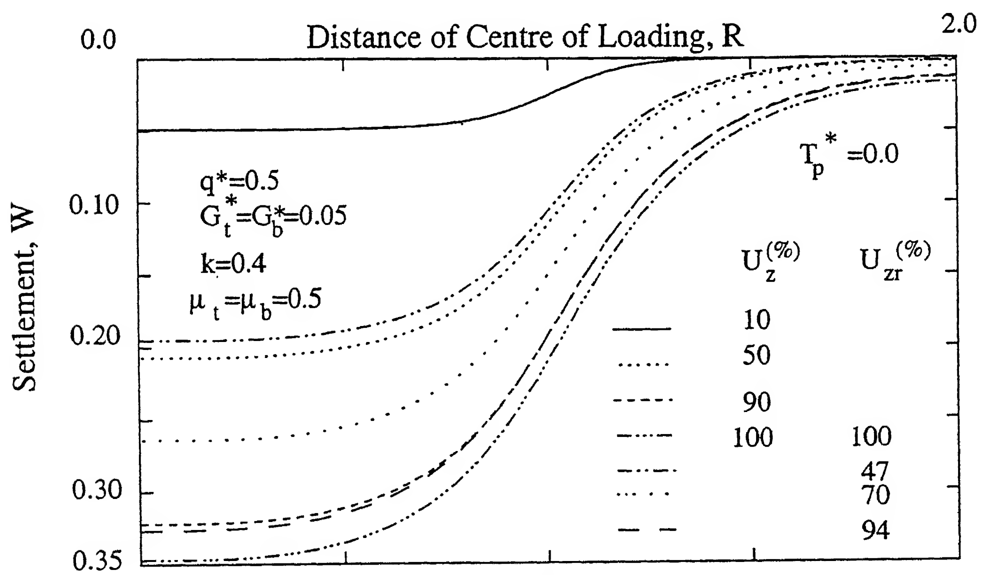

Fig. 4.12. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.5)

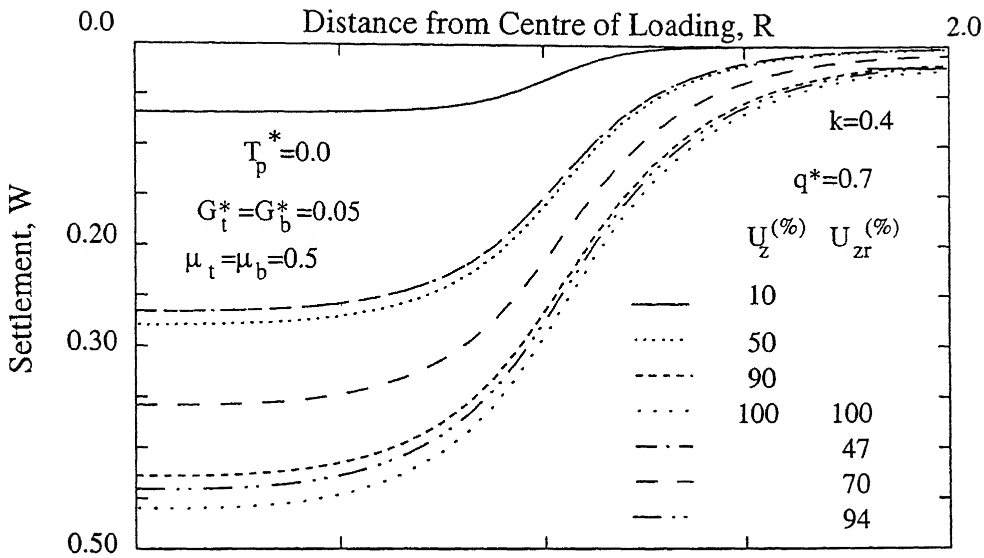

Fig. 4.13. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for q*=0.7)

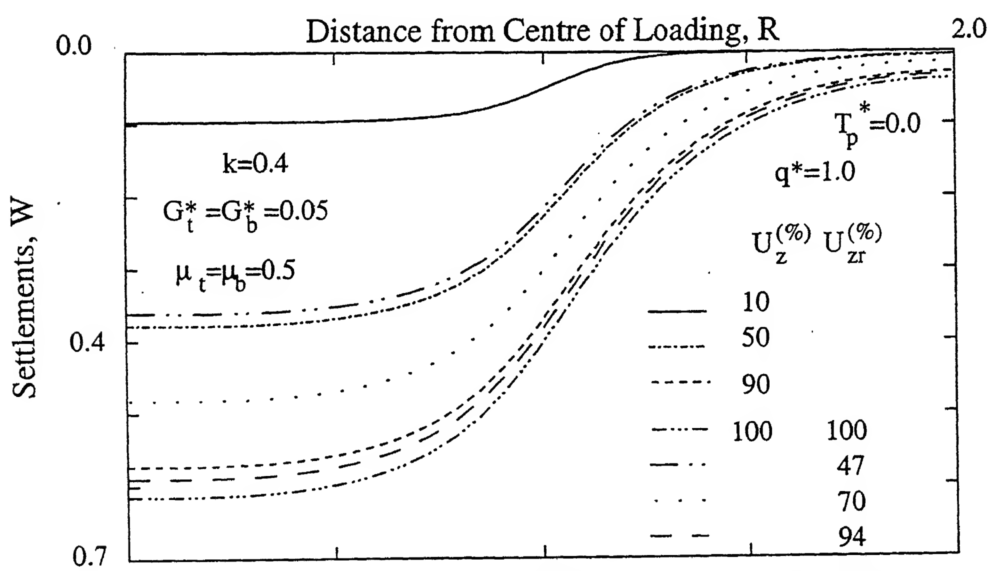

Fig. 4.14. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=1.0)

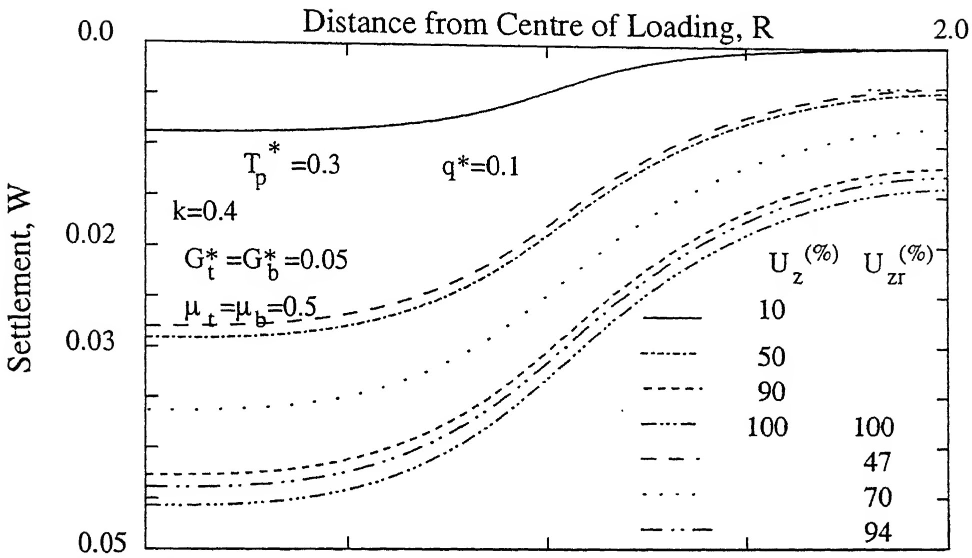

Fig. 4.15. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.1)

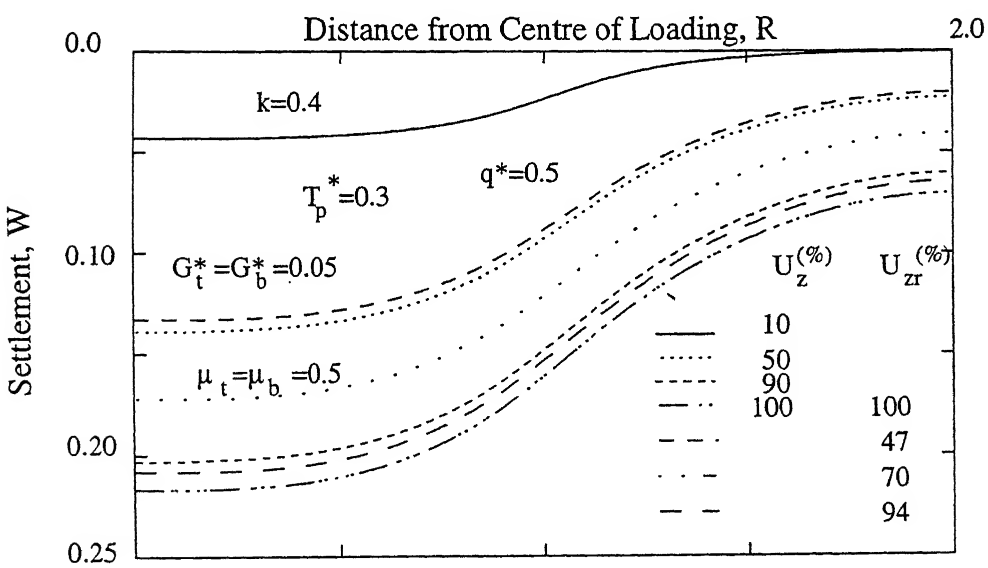

Fig. 4.16. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.5)

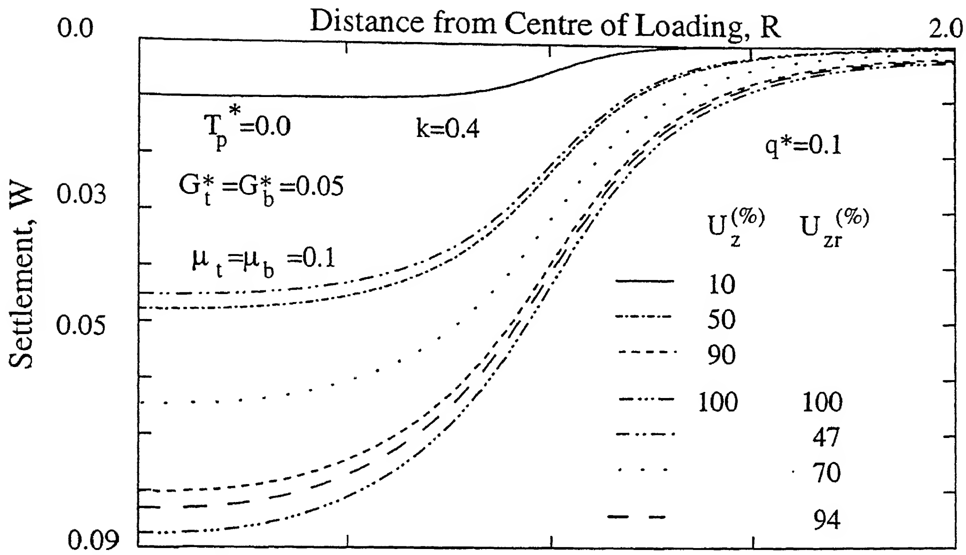

Fig. 4.19. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.1)

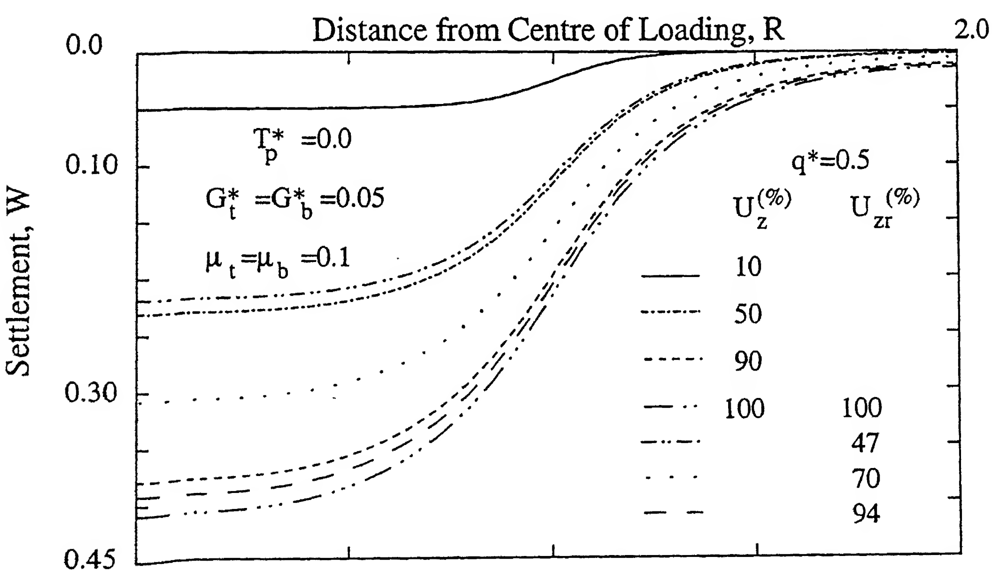

Fig. 4.20. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.5)

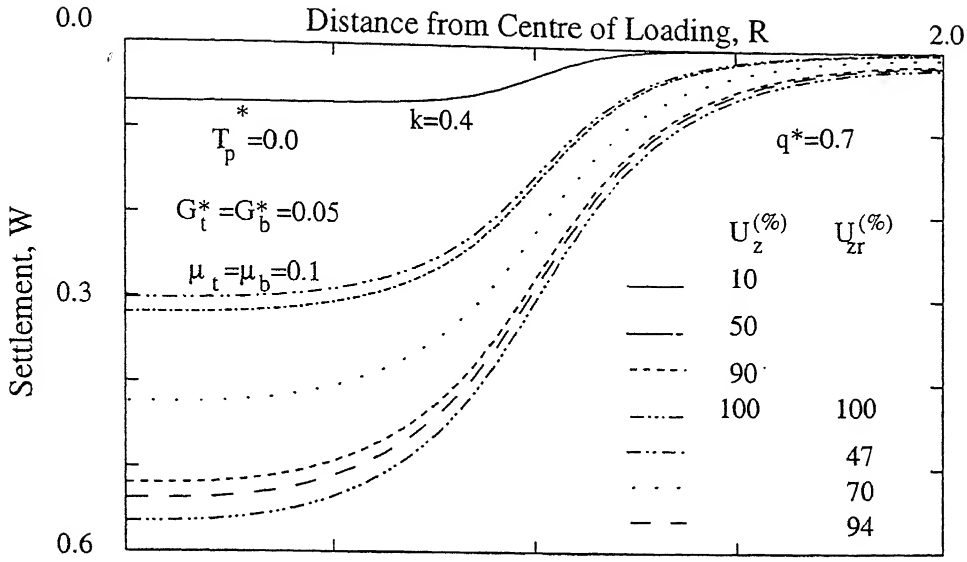

Fig. 4.21. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.7)

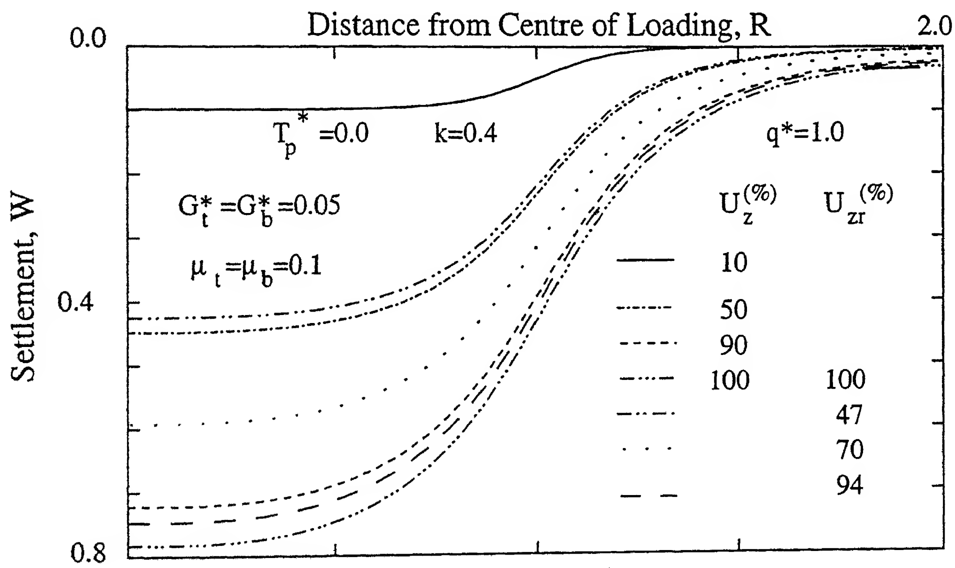

Fig. 4.22. Load-settlement profiles at varuious stages of consolidation of soft foundation soil (for, q*=1.0)

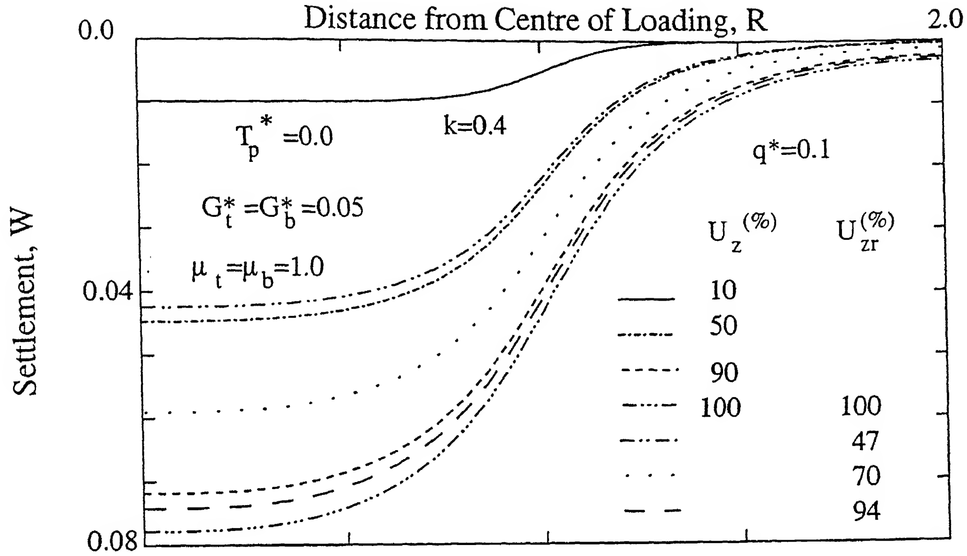

Fig. 4.23. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.1)

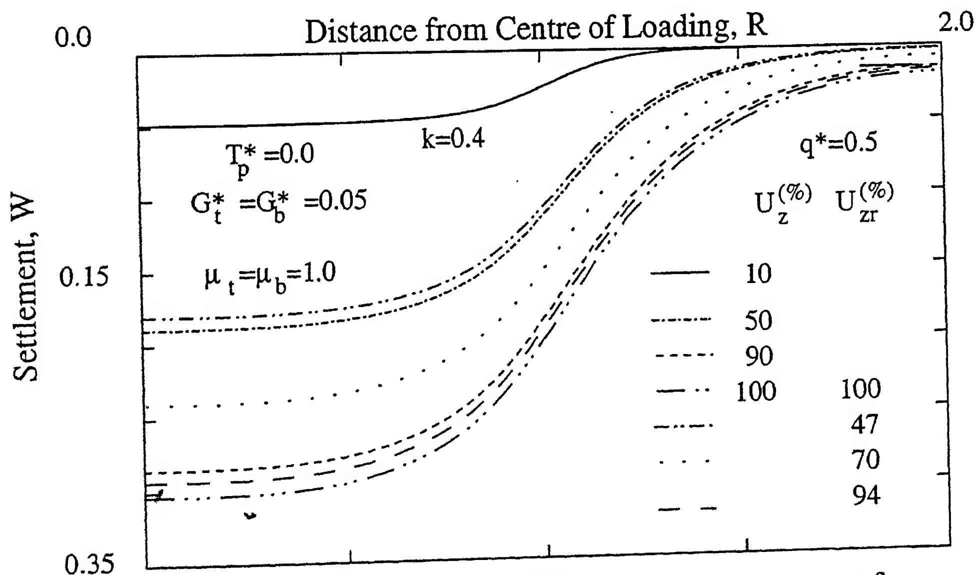

Fig 4.24. Load-settlement profiles at various stages of consolidation of the soft foundaion soil (for, q*=0.5)

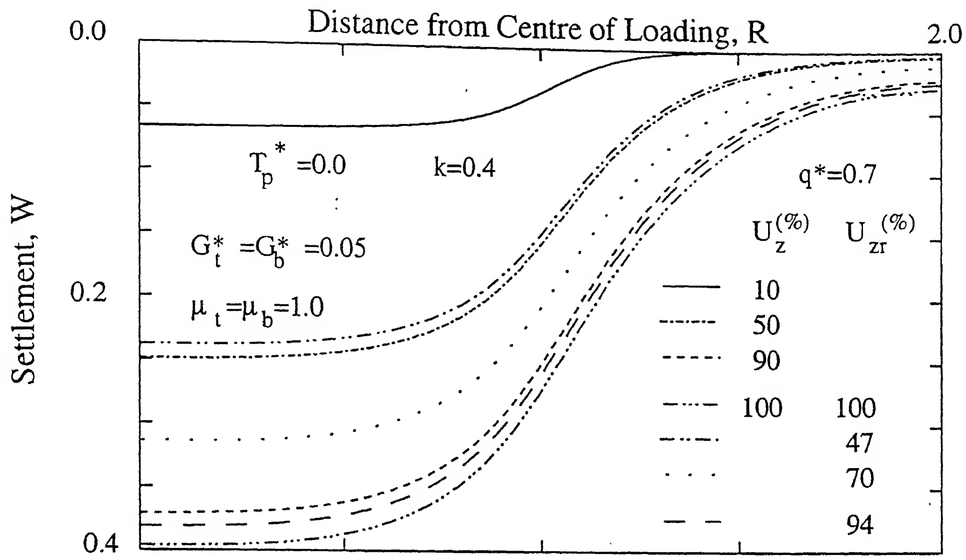

Fig. 4.25. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.7)

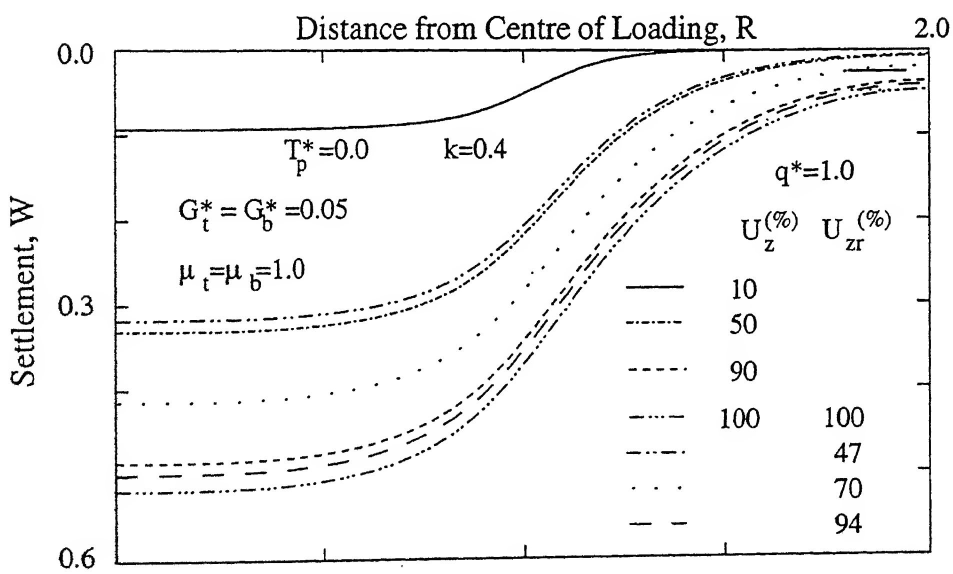

Fig. 4.26. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=1.0)

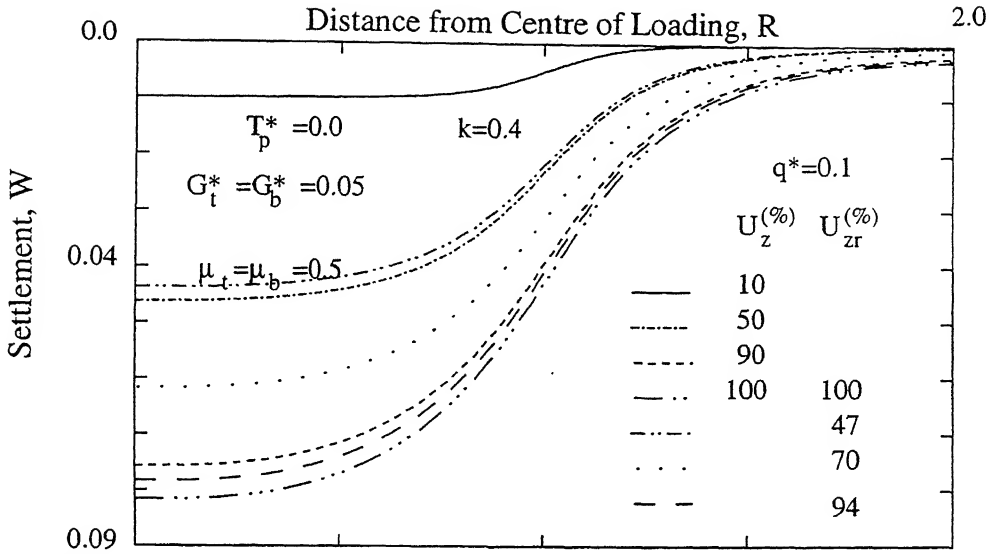

Fig 4.27 . Load-settlement profiles at various stages of consolidation on the soft foundation soil (for, q*=0.1)

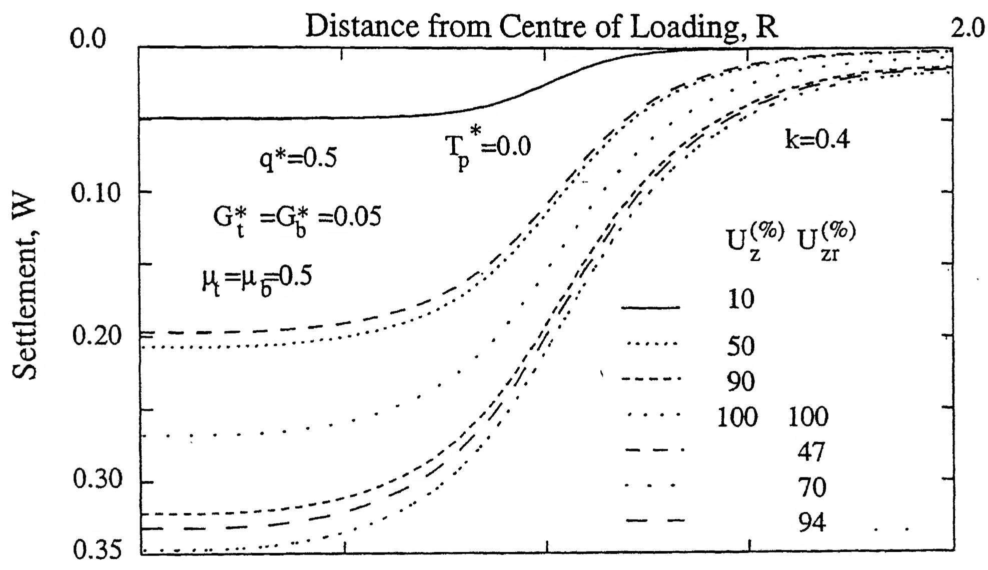

Fig. 4.28. Load-settlements profiles at various stages of consolidation of the soft foundation soil (for, q*=0.5)

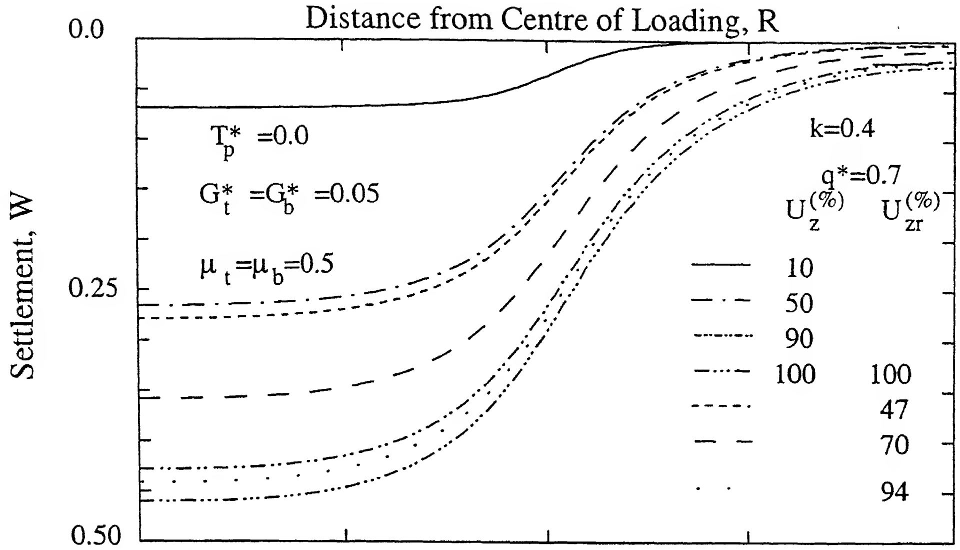

Fig. 4.29. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.7)

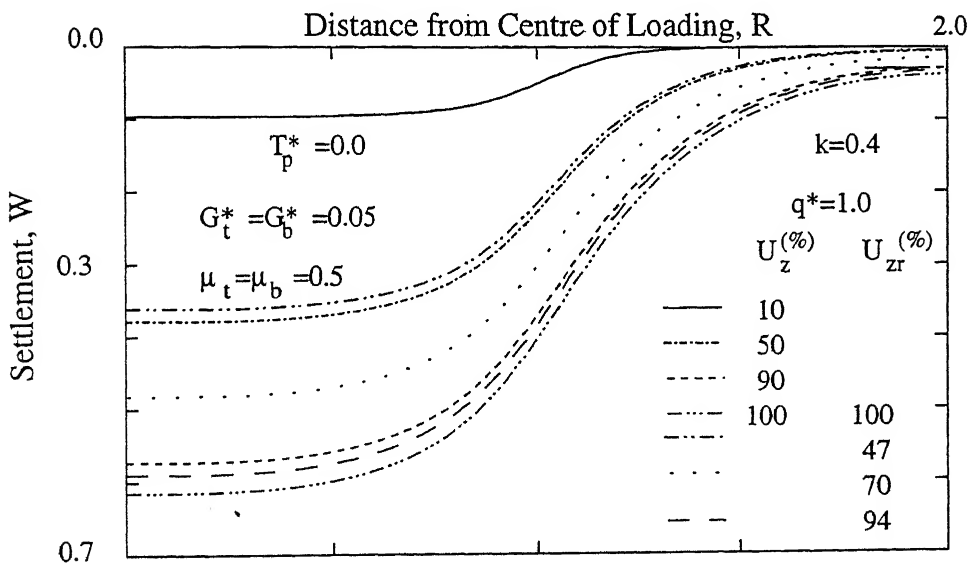

Fig. 4.30. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=1.0)

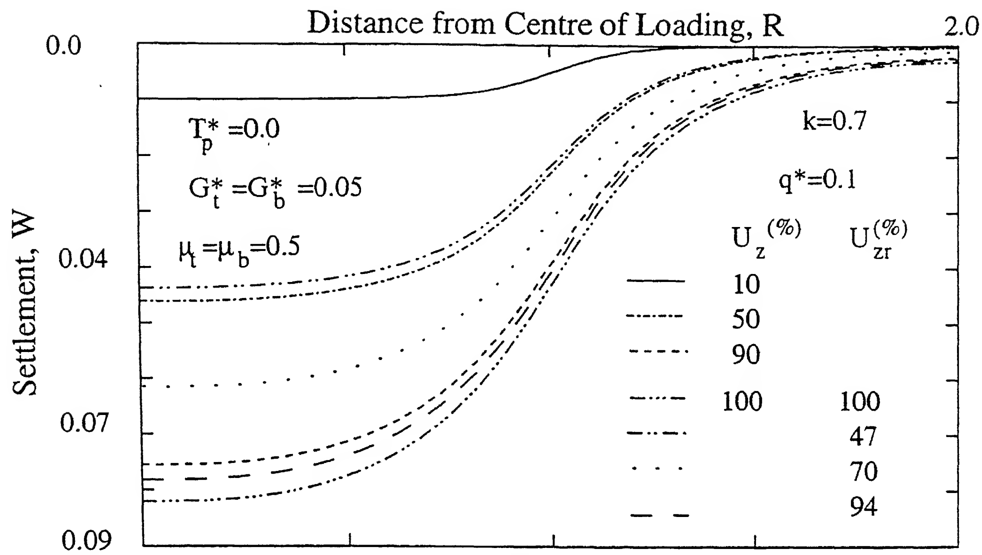

Fig. 4.31. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.1)

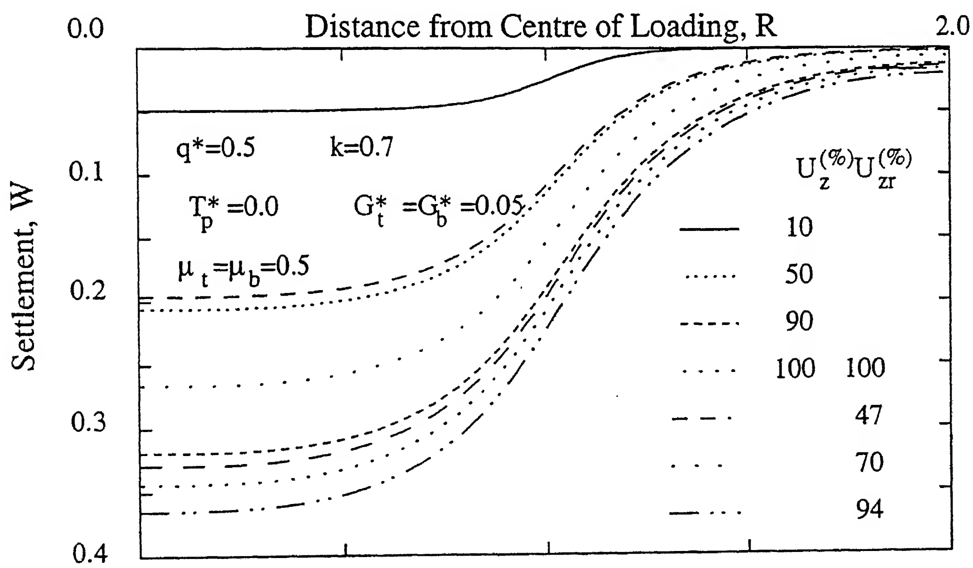

Fig. 4.32. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.5)

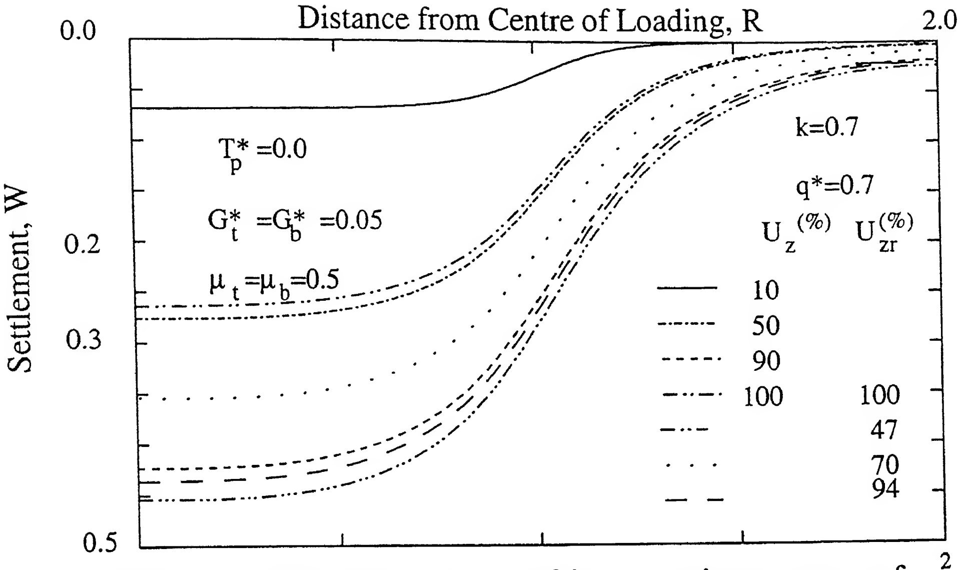

Fig. 4.33. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.7)

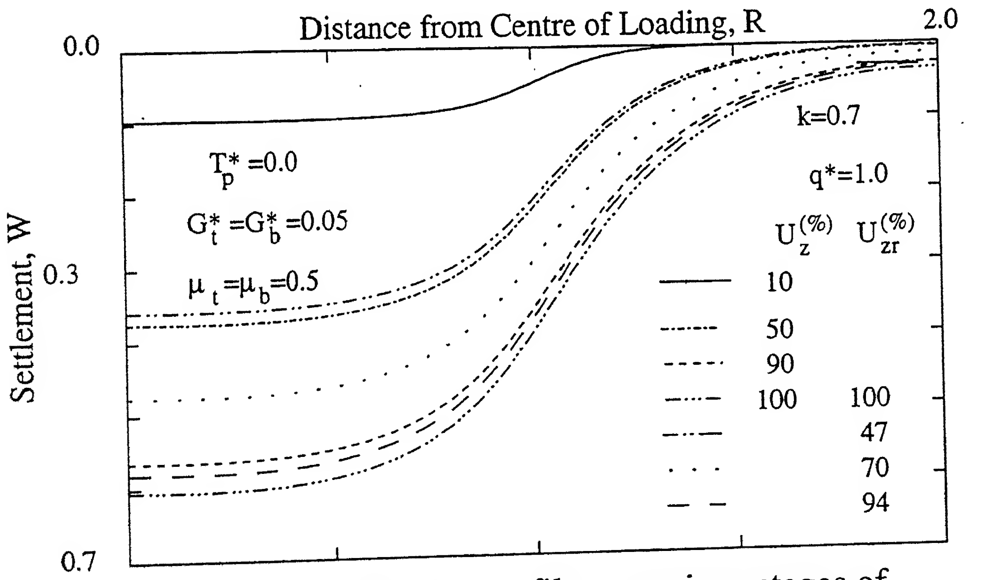

Fig. 4.34. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=1.0)

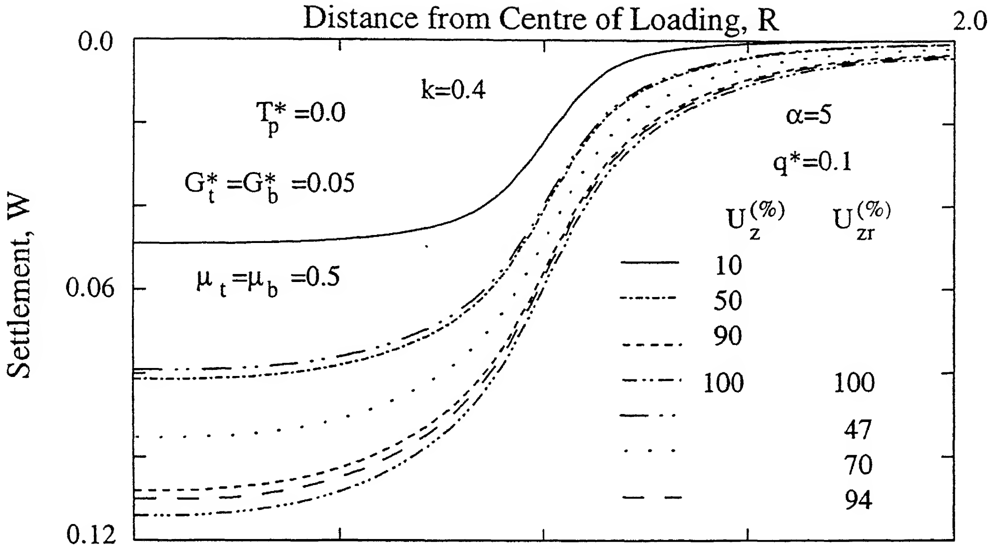

Fig. 4.35. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.1)

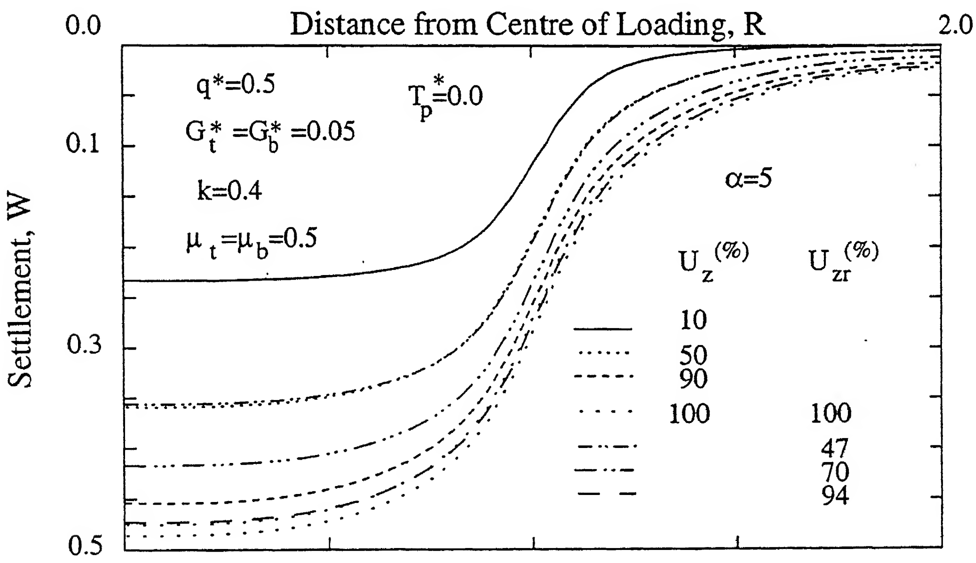

Fig. 4.36. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.5)

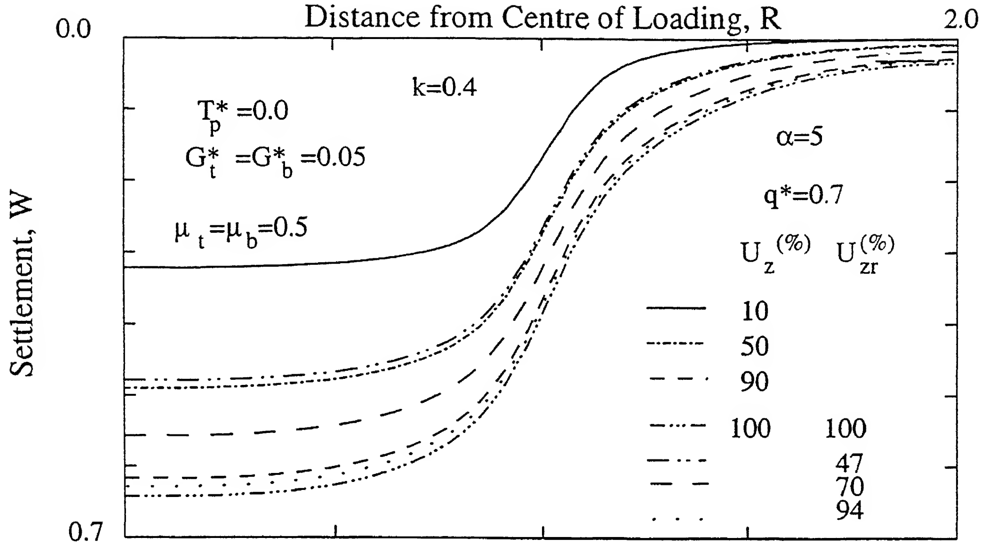

Fig. 4.37. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.7)

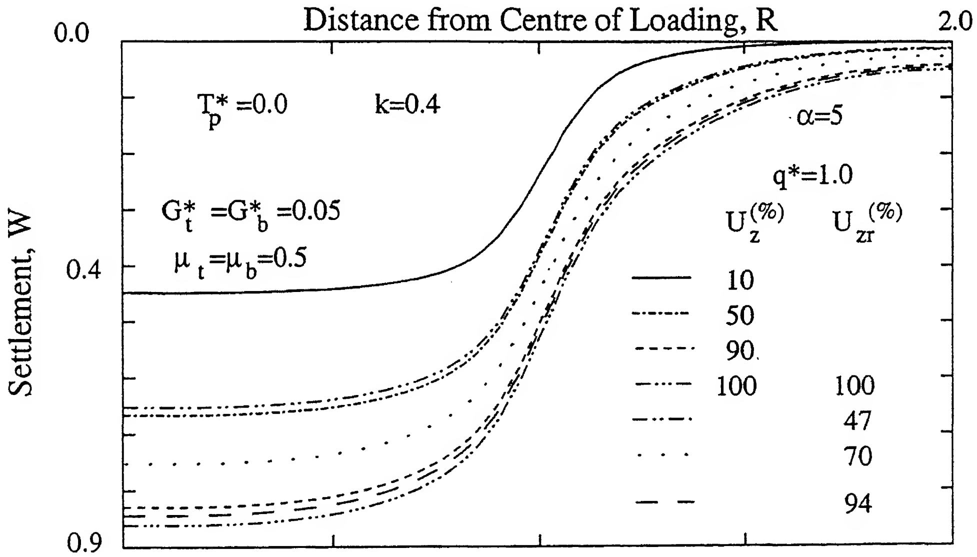

Fig. 4.38 Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=1.0)

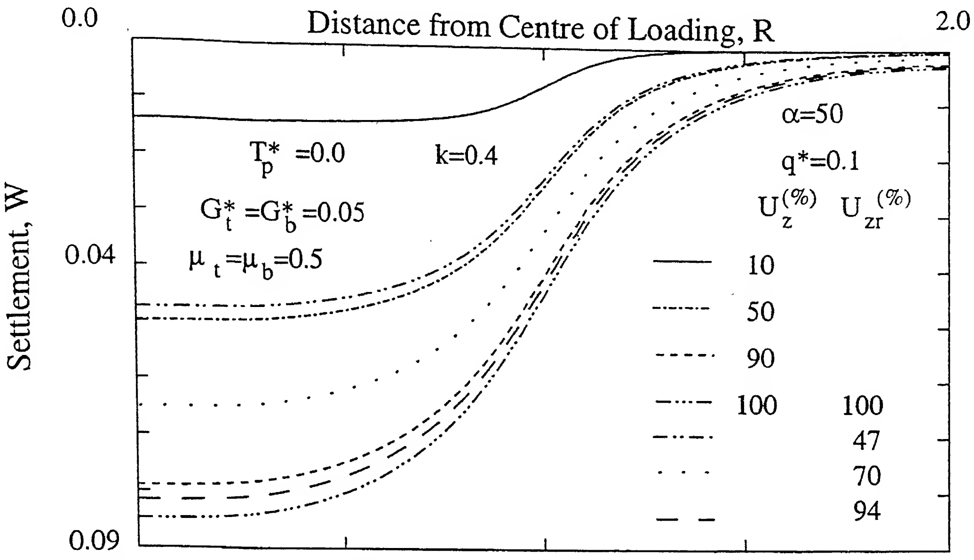

Fig 4.39. Load-settlement profiles at various stage of consolidation of the soft foundation soil (for, q*=0.1)

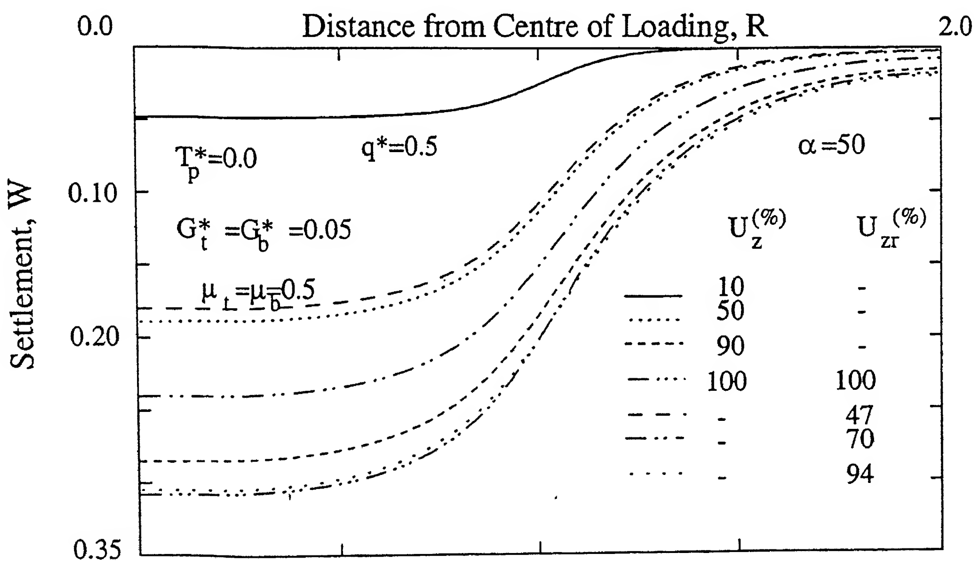

Fig. 4.40. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.5)

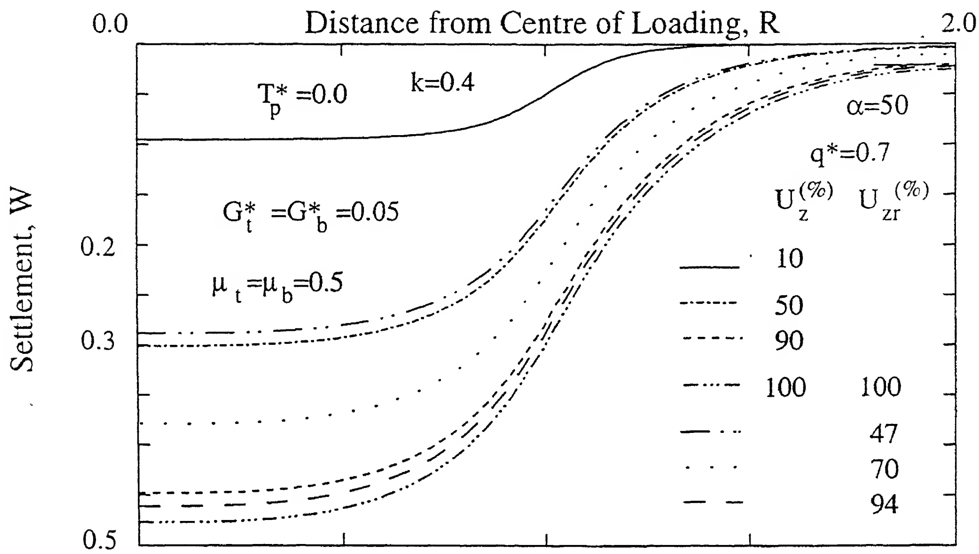

Fig 4.41 Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=0.7)

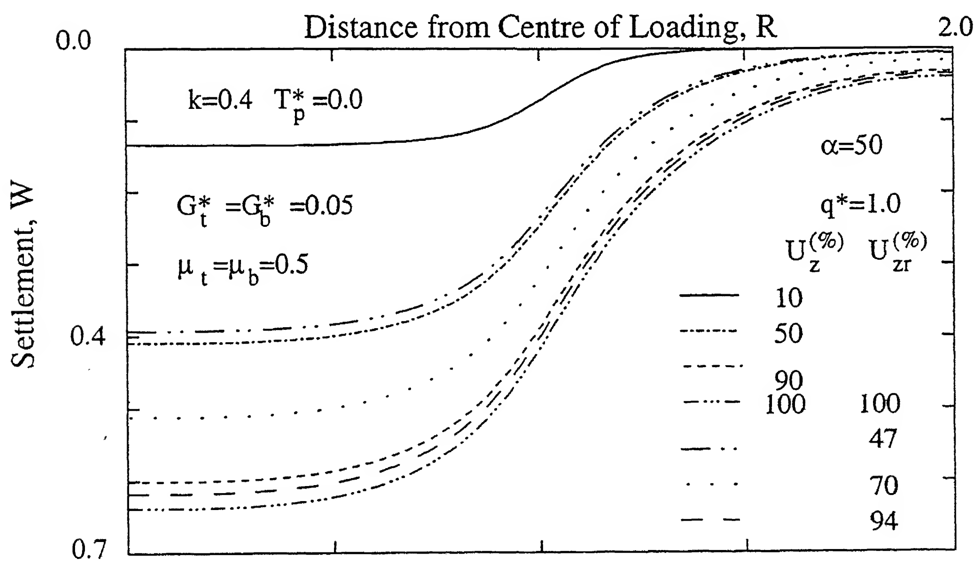

Fig 4.42. Load-settlement profiles at various stages of consolidation of the soft foundation soil (for, q*=1.0)

CHAPTER 5
SUMMARY AND CONCLUSIONS

The reinforcement of a granular fill on a soft foundation soil with geosynthetics is growing its popularity nowadays. To identify the behaviour of this, there are several models proposed and are reported in literature.

In the present work, an improved mechanical model is proposed to obtain the settlement response of the model considering three-dimensional consolidation of the soft foundation soil. Considering the equilibrium of different elements under the applied load, the equations governing the response function of the model are derived for axi-symmetric case. The governing equations in their nondimensional forms are solved iteratively by finite difference method and the results are presented in nondimensional form, which are discussed in chapter 4.

Based on the results and discussions presented in chapter 4, the following generalized conclusions are drawn:

1. The settlement response obtained by the proposed model for one-dimensional case is shown to be in reasonably good agreement with the results reported in literature.

2. The ultimate settlement at the end of the consolidation process of the soft foundation soil of geosynthetic-reinforced granular fill-soft soil foundation system is independent of one-dimensional or three-dimensional consolidation methods.

3. The difference in settlement response under one-dimensional and three-dimensional consolidation is more at lower loads. The difference gradually reduces as the load intensity increases.

4. The difference in settlement response under one-dimensional and three-dimensional consolidation is more at the initial stages of consolidation. As the time progresses, the settlement increases; but the difference in settlement reduces significantly and finally it vanishes.

5. There are several parameters e.g. prestress in geosynthetic reinforcement, interfacial friction coefficients, compressibility, shear parameters and lateral stress ratio of the granular fill etc. which may have significant effect on the settlement behaviour under different degrees of consolidation.

6. The difference in settlement response under different degrees of consolidation is more significant at the centre of the loaded region as well as at the edges for high value of shear parameters.

7. The prestress force on the geosynthetic reinforcement shows the significant changes in settlement response at the centre of the loaded region as well as at the edges. The difference in settlement response while considering three-dimensional consolidation as expected is higher at the initial stages of consolidation and at lower loads.

8. The effect of interfacial friction of the membrane shows the usual trend of settlement response. The difference is higher at lower loads and initial stages of consolidation.

9. The effect of lateral stress ratio of the granular fill shows significant changes in the settlement response at the centre of the loaded region at lower loads and at the initial stages of consolidation. No significant effect has been observed at the edges.

10. The change in settlement response under the different degrees of consolidation is more at high value of modular ratio. So, the effect of compressibility plays a significant role on the settlement response at the centre of the loaded region.

11. The numerical approach is found to be very efficient in terms of economy of computation time and takes only few seconds of CPU time to obtain the settlement in the geosynthetic reinforcement.

RECOMMENDATIONS FOR FURTHER WORK

1. Extension of proposed foundation model to incorporate

   (i) nonlinearity of fill and soft subgrade

   (ii) plasticity of fill and soft subgrade

   (iii) variation of k with the depth of the granular fill

   (iv) variation of modulus of subgrade reaction with depth of the foundation soil and also with time

   (v) relative slip at fill-geosynthetic interface.

2. Validation of the obtained results through laboratory and field model tests.

3. Improvement of solution technique.

REFERENCES

Aboshi, H. (1984). Soil Improvement Techniques in Japan. *Proc. Seminar on Soil Improvement and Construction Techniques in Soft Ground,* Singapore, 3-16.

Akimusuru, J.O., and Akinbolade, J.A. (1981). Stability of Loaded Footings on Reinforced Soil. *J. Of Geotechnical Engg. Div., ASCE,* **107,** 819-827.

Al-Hussaini, N.N. and Townsend, F.C.(1975) Investigation of k_0 testing in Cohesionless Soils. Technical Report S-75-11, *US Army Engineering, Waterways Station,* Vicksburq, Mississippi, 70.

Alpan (1967). The Empirical Evaluation of the Coefficient k_0 and k_{0R}. *Soil and Foundation,* 7, 31-40.

Athanasopoulos, G.A., (1996). Results of Direct Shear Tests on Geosynthetic Reinforced Cohesive Soil. *Geotextiles and Geomembranes,* **14,** 619-644.

Barvashov, V.A., Budanov, V.G., Fomin, A.N., Perkov, U.R., and Pushkin, V.I. (1977). Deformations of Soil Foundations with Prestressed Synthetic Fabrics. *C.R.Coll. Int. Soils Textiles,* Paris.

Basset, R.H., and Last, N.C. (1978). Reinforcing Earth Below Footings and Embankments. *Proc. Symp. Earth Reinforcement,* ASCE, Pittsburgh, 202-231.

Binquet, A.M., and Lee,K.L.(1975a) Bearing capacity Analysis of Reinforced Earth Slabs. *J.of Geotech. Engg. Div.,* ASCE, **101,** 1257-1276.

Binquet, A.M., and Lee. K.L. (1975b) Bearing capacity tests on Reinforced Earth Slabs. *J. Of Geotech. Engg. Div.,* ASCE, **101,** 1241-1255.

Bourdeau, P.L., Harr, M.E., and Holltz, R.D.(1982) Soil-Fabric Interaction –an Analytical Model. *Proc. 2nd Int. Conf. On Geotextiles,* Vol. 2, Las Vegas, USA,387-391.

Bourdeau, P.L. (1989). Modelling of Membrane Action in a two-Layer Reinforced Soil System. *Computer and Geotechnics,* 7, 19-36.

Broms, B.(1971). Lateral Earth Pressure due to Compaction of Cohesionless Soils. *Proc. 4th Conf. On Soil Mechanics,* Budapest, 373-384.

Broms, B. (1987). Stabilization of Very Soft Clay using Geofabric. *Geotextiles and Geomembranes,* **5**, 17-28.

Brown, B.S and Poulos, H.G., (1981) Analysis of Foundation on Reinforced Soil. *10th Int. Conf. On Soil Mechanics and Foundation Engg.*, Vol. 3, Stockholm, 595-598

Das, B.M. (1989). Foundation on Sand Underlain by Soft Clay with Geotextile at Sand-Clay Interface. *Proc. Of Geosynthetics' 89 Conference,* San Deago, USA, 203-213.

Das, B.M. and Shin, E.C. (1994). Strip Foundation on Geogrid-Reinforced Clay: Behaviour under Cyclic Loading. *Geotextiles and Geomembranes,* **13,** 657-667.

Das, B.M., Maji, A.and Shin, E.C. (1998) Foundation of Geogrid-Reinforced Sand-Effect of Transient Loading. *Geotextiles and Geomembranes,* **16,** 151-200.

Dembicki, E., Jermolowicz, P. and Niemunism, A. (1986). Bearing capacity of Strip Foundation on Soft Soil Reinforced by Geotextiles. *Proc. 3rd Int. Conf. On Geotextiles,* Vienna, 205-209.

Dixit, R.K. and Mandal, J.N. (1993). Bearing capacity of Geosynthetic-Reinforced Soil Using Variational Method. *Geotextiles and Geomembranes*, **12,** 543-566.

Duncan, J.M. and Seed, R.B. (1986) Compaction-Induced Earth Pressures Under k_0 Conditions. *J. Of Geotech. Engg. Div.,* ASCE, **112,** 1-22.

Espinoza, R.D.(1994). Soil-Geotextile Interaction: Evaluation of Membrane Support. *Geotextiles and Geomembranes,* **13,** 281-293.

Fragaszy, R.J. and Lawton, E. (1984). Bearing capacity of Reinforced Sand Subgrades. *J. Of Geotech. Engg. Div.,* ASCE, **110,** 1500-1506.

Ghosh, C.(1991). *Modelling and Analysis of Reinforced Foundation Beds.* Ph.D Thesis, Dept. of Civil Engg, Indian Institute of Technology, Kanpur, India

Ghosh, C. and Madhav, M.R. (1994). Settlement Response of a Reinforced Shallow Earth Bed. *Geotextiles and Geomembranes,* **13,** 643-656.

Giroud, J.P. and Noiray, L. (1981). Geotextiles-Reinforced Unpaved Road Design. *J. Of Geotech. Engg. Div.,* ASCE, **107,** 1223-1254.

Guido, V.A., Chang, D.K. and Sweenay, M.A. (1986) Comparison of Geogrid and Geotextile Reinforced Earth Slabs. *Can. Geotech. J.,* **23,** 435-440.

Harrison, W.J. and Gerrard, M. (1972) Elastic Theory Applied to Reinforced Earth. *J. Of Soil Mechanics and Foundations Div.,* ASCE, **98,** 1325-1345.

Hausmann, M.R. (1990). *Engineering Principles of Ground Modification.* McGraw-Hill, New York.

Ingold, T.S. and Miller, K.S. (1982). Analytical and Laboratory Investigations of Reinforced Clay. *Proc. 2nd Int. Conf. On Geotextiles,* Las Vegas, Vol. 3, 587-592

Jaky, J. (1994). The Coefficient of Earth Pressure at Rest. *J. Soc. Hung. Arch. And Engg.,* **8,** 355-388.

Kerr, A.D. (1964) Elastic and Viscoelastic Foundation Models. *J. Of Applied Mechanics,* **31,** 491-498.

Khing, K.H., Das, B.M., Puri, B.K., Yen, S.C. and Cook, E.E. (1994) Foundation on Strong Sand Underlain by Weak Clay with Geogrid at the Interface. *Geotextiles and Geomembranes,* **13,** 199-206.

Kim, S.I. and Cho, S.D. (1988) An Experimental Study on the Contribution of Geotextiles to Bearing capacity of Footings on Weak Clays. *Int. Geotech. Symp. On Theory and Practice of Earth Reinforcement,* Fukuoka, Japan, 215-220.

Kinny, T. (1982). Small Scale Load Tests on a Soil-Geotextile-Aggregate System. *Proc. 2nd Int. Conf. On Geotextiles,* Las Vegas, USA, 405-409.

Koerner, R.M. (1990). *Designing with Geosynthetics.* Prentice Hall, Englewood Cliffs, NJ, USA.

Koga, K., Aramaki, G. and Valliappan, S. (1988). Finite Element Analysis of Grid Reinforcement. *Int. Geotech. Symp. On Theory and Practice of Earth Reinforcement,* Fukuoka, Japan, 407-411.

Love, J.P., Burd, H.J., Milligan, G.W.E.and Houlsby, G.T. (1987). Analytical and Model Studies of Reinforcement of a Layer of Granular Fill on a Soft Clay Subgrade. *Can. Geotech. J.,* **24,** 611-622.

Madhav, M.R. and Poorooshasb, H.B. (1988). A New Model for Geosynthetic Reinforced Soil. *Computer and Geotechnics,* **6,** 277-290.

Madhav, M.R. and Poorooshasb, H.B. (1989). Modified Pasternak Model for Reinforced Soil. *Math and Computer Modelling, An Int. J.,* **12,** 1505-1509.

Madhav, M.R and Ghosh, C. (1990). Settlement of Geosynthetic Reinforced Ground. *Proc. Int. Conf. On Geotextiles, Geomembranes and Related Products,* Hauge, Netherlands, 163.

Mandal, J.N. and Sah, H.S. (1992). Bearing capacity Tests on Geogrid-Reinforced Clay. *Geotextiles and Geomembranes,* **11,** 327-333.

Mandal, J.N. and Joshi, A.A. (1996). Design of Geosynthetic Reinforced Embankments on Soft Soils. *Geotextiles and Gemembranes,* **14**, 137-145.

Manjunath, V.R. and Dewaikar, D.M. (1994). Model Footing Tests on Geofabric Reinforced Granular Fill Overlying Soft Clay. *Proc. 5th Int. Conf. On Geotextiles, Geomembranes and Related Products,* Vol. **1,** Singapore, 327-330.

Matsuoka, H. (1976). On the Significance of the Spatial Mobilized Plane. *Soils and Foundations,* **16,** 91-100.

McGown, A., Andrawes, K.Z., Werner, G. and Yusuf, Z. (1990). Bending and Tensile Reinforcement of Unbound Roads. *Proc. Int.Conf. On Geotextiles, Geomembranes and Related Products,* Hauge, Netherlands, 239-246.

Nishida, K. and Nishigata, T. (1994). The Evaluation of Separation Function for Geotextile. *Proc. 5th Int. Conf. On Geotextiles, Geomembranes and Related Products,* **Vol. 1,** Singapore, 139-142.

Omar, M.T., Das, B.M., Puri, V.K. and Yen, S.C. (1993). Ultimate Bearing capacity of Shallow Foundations on Sand with Geogrid Reinforcement. *Can. Geotech. J.*, **30,** 545-549.

Otani, J., Ochiai, H., Miyata, Y. (1994). Bearing capacity of Geogrid Reinforced Grounds. *Proc. 5th Int. Conf. On Geotextiles, Geomembranes and Related Products,* Singapore, **Vol. 1,** 117-120.

Poorooshasb, H.B. (1989). Analysis of Geosynthetic Reinforced Soil Using a Simple Transform Function. *Computers and Geotechnics,* **8,** 289-309.

Poorooshasb, H.B. (1991a). Load Settlement Response of a Compacted Fill Layer Supported by a Geosynthetic Overlying a Void. *Geotextiles and Geomembranes,* **10,** 179-201.

Poorooshasb, H.B. (1991b). On Mechanics of Heavily Reinforced Granular Materials. *Soils and Foundations,* **31,** 134-152.

Puri, V.K., Yen, S.C., Das, B.M. and Yeo, B. (1993) Cyclic Load-Induced Settlement of a Square Foundation on Geogrid-Reinforced Sand. *Geotextiles and Geomembranes,* **12,** 587-597.

Rowe, R.K. and Soderman, K.L. (1987). Stabilization of Very Soft Soils Using High Strength Geosynthetics: The Role of Finite Element Analysis. *Geotextiles and Geomembranes,* **6,** 53-80.

Sakti, J.P. and Das, B.M. (1987). Model Tests for Strip Foundation on Clay Reinforced with Geosynthetic Layers. *Transportation Research Records,* No. 1153, 40-45.

Sawicki, A. (1983). Plastic Limit Behaviour of Reinforced Earth. *J. Of Geotech. Engg.* ASCE, **109,** 1000-1005.

Schlosser, F. and Long, N.T.(1972). Comportement de la terre armee dans les ouvrages de soutenement. *Proc. 5th Eoropean Conf. On Soil Mechanics. And Foundations,* **Vol. 1,** 299-306.

Sellemeijer, J.B. (1990). Design of Geotextile Reinforced Unpaved Roads and Parking Areas. *Proc. 4th Int. Conf. On Geotextiles, Geomembranes and Related Products,* **Vol. 1,** Hauge, Netherlands, 177-182.

Selvadurai, A.P.S.(1979)*Elastic Analysis of Soil-Foundation Interaction.* Elsevier Sc. Publ. Co. Amsterdam.

Selvadurai, A.P.S. and Gnanendran, C.T. (1989). An Experimental Study of a Footing Located on a Sloped Fill: Influence of a Soil Reinforcement Layer. *Can. Geotech. J*, **26,** 467-473.

Shukla, S.K. and Chandra, S. (1994). A Generalized Mechanical Model for Geosynthetic Reinforced Foundation Soil. *Geotextiles and Geomembranes,* **13,** 813-825.

Shukla, S.K. (1995). *Foundation Model for Reinforced Granular Fill-Soft Soil System and its Settlement Response.* Ph.D. Thesis, Dept. of Civil. Engineering, Indian Institute of Technology, Kanpur, India.

Sowers, G.F., Collins, S.A. and Miller, D.G. (1982). Mechanism of Geotextile-Aggregate Support in Low-Cost Roads. *Proc. 2nd Int. Conf. On Geotextiles,* Las Vegas, USA, 341-346.

Terzaghi, K. (1943). *Theoretical Soil Mechanics.* John Wiley & Sons. Inc. New York.

Valsangkar, A.V. and Char, A.N.R. (1987). Model Tests on Peat-Geotextile-Lightweight Aggregate System. *Geotextiles and Geomembranes,* **5,** 251-260.

Vidal, H. (1969). The Principle of Reinforced Earth. *Highway Research Record* No. 282.

Vokas, C.A. and Stoll, R.D. (1989). Reinforced Elastic Layered Systems. *Transportation Research Record,* No. 1153, 1-7.

Westergaard, H.M. (1938). *A Problem of Elasticity Suggested by a Problem in Soil Mechanics.* Mechanics of Solid, Timoshenko 60th Anniversary, Vol., Macmillan & Co. New York.

Wu, J.T.H., Siel, B.D., Chou, N.N.S. and Helwany, H.B. (1992). The Effectiveness of Geosynthetic Reinforced Embankments Constructed over Weak Foundations. *Geotextiles and Geomembranes,* **11,** 133-150.

Yang, Z. (1972). *Strength and Deformation Characteristics of Reinforced Sand.* Ph.D. Dissertation, University of California at Los Angeles.

Yetmoglu, T., Wu. J.T.H. and Saglamer, A. (1994). Bearing capacity of Rectangular Footings on Geogrid-Reinforced Sand. *J. Of Geotech. Engg. Div.,*ASCE,**120,**2083-2099.

Zhao, A.(1996). Failure Load on Geosynthetic Reinforced Soil Structures.*Geotextiles and Gemembranes,***14,** 289-300.

A 130459

A 130459

Date Slip

This book is to be returned on the date last stamped.

A130459